CNWRA 93-007

SEISMIC/TORNADO ANALYSIS REVIEW FOR THE VITRIFICATION FACILITY AT WEST VALLEY

Prepared for

Nuclear Regulatory Commission
Contract NRC-02-88-005

Prepared by

Center for Nuclear Waste Regulatory Analyses
San Antonio, Texas

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San Antonio, Texas

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ACKNOWLEDGMENTS

This report was prepared to document work performed by the Center for Nuclear Waste Regulatory Analyses (CNWRA) for the U.S. Nuclear Regulatory Commission (NRC) under Contract No. NRC-02-88-005. The activities reported here were performed on behalf of the NRC Office of Nuclear Material Safety and Safeguards (NMSS), Division of Fuel Cycle Safety and Safeguards. The report is an independent product of the CNWRA and does not necessarily reflect the views or regulatory position of the NRC.

1 INTRODUCTION

The purpose of this report is to review the seismic and tornado analyses conducted by the U.S. Department of Energy (DOE) and their contractors for assessing the integrity of the Vitrification Facility (VF) at West Valley. The DOE analyses were conducted over a period of several years, with several interactions between the DOE and the U.S. Nuclear Regulatory Commission (NRC) on various aspects of the seismic- and tornado-related designs. In the past, the NRC has raised questions about the preliminary analyses presented. In response to the questions, the DOE has developed additional analyses and documented the calculations. The review presented here has integrated the NRC comments raised earlier and DOE's responses to them. Thus, this review provides an assessment of the degree of resolution of the issues. Additional issues have been identified in the current review that can be addressed by the DOE, as part of the Safety Analysis Report (SAR) on the VF to be developed in FY 94-95.

The analyses have been reviewed to determine the adequacy of the prediction of responses of safety-related structures to seismic and tornado loading. The predicted responses will be important to prediction of their effects on radiological safety, but magnitude of radiological release and prediction of effects on radiological safety are not within the scope of this report. The purpose of this review and assessment is to define outstanding open issues and additional analyses in the VF SAR. The design criteria used by the DOE for the seismic and structural analysis conforms to engineering practices for other nuclear facilities. The review has been conducted using American Concrete Institute (ACI) and American Institute of Steel Construction (AISC) codes, DOE orders, and NRC Regulatory Guides (RG).

This report is not a Safety Evaluation Report (SER) since the SAR, which would be reviewed by such an SER, is not expected until FY 94-95. This report will be referenced as part of the SER on the VF SAR.

The Waste Solidification Systems (WSS) Program Element at the Center for Nuclear Waste Regulatory Analyses (CNWRA) has several technical tasks supporting the NRC need in its review capacity of the West Valley Demonstration Project (WVDP) which is being conducted by the DOE.

The WVDP is being undertaken by the DOE pursuant to the WVDP Act (P.L. 96-368). The Act directs the WVDP to five major activities, as follows:

- Solidify the liquid high-level waste (HLW) stored at the site
- Develop containers for the solidified HLW
- Transport the waste to a federal repository for disposal
- Dispose of low-level waste (LLW) and transuranic (TRU) waste produced during the project
- Decontaminate and decommission facilities.

It is the task of WVDP to convert the HLW into borosilicate glass for ultimate disposal in a federal repository. Figure 1-1 is a schematic of the process that will be used to transform the HLW to the borosilicate glass form. The WVDP operations are controlled by DOE directives, with oversight

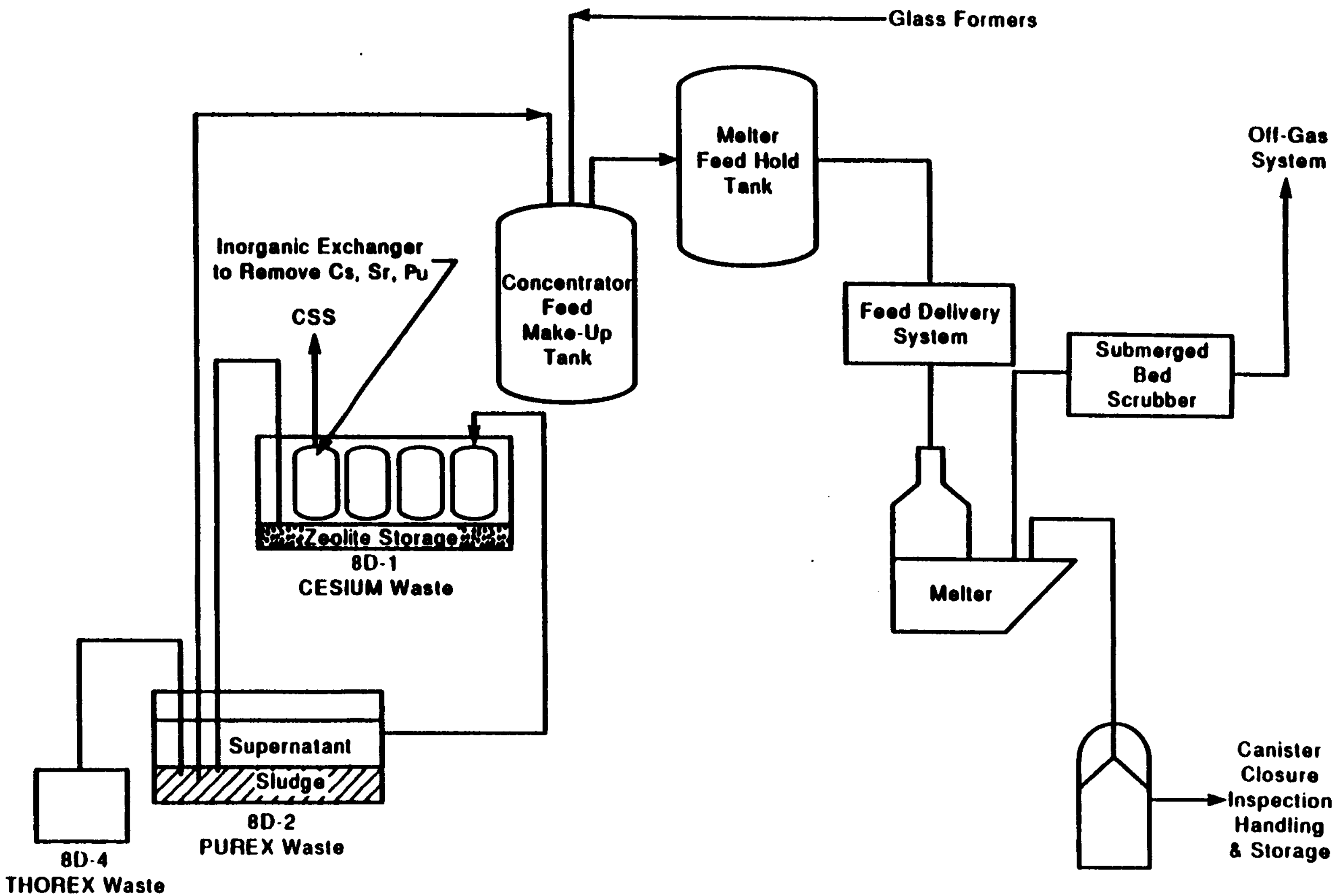


Figure 1-1. High-level waste processing flow diagram (Tschoepe et al., 1991)

provided by the NRC to ensure no significant risk to public health and safety. The WVDP site is located on New York state's western plateau, near the community of West Valley, about 55 km south of Buffalo in the town of Ashford, Cattaraugus County.

The West Valley site was a commercial nuclear fuel reprocessing center that operated from 1966 to 1972. The DOE, together with its prime contractor, Westinghouse Electric Corporation, officially took over site operation in February 1982. The DOE was mandated by Congress to carry out a high-level waste (HLW) management demonstration project at this site. This project involves the cleanup of over 2.1 million liters of nuclear spent fuel reprocessing wastes. The HLW terminal form will be borosilicate glass. The HLW is stored in two tanks containing two different waste forms. Alkaline Plutonium-Uranium Recovery EXtraction (PUREX) wastes are stored in Tank 8D-2 (approximately 2.1 million liters) in two distinct phases, referred to as the supernatant (the liquid phase) and the sludge (the solid phase). Acidic THOrium Recovery EXtraction (THOREX) wastes are stored in Tank 8D-4 (approximately 31,000 liters), in essentially a single, liquid phase. The vitrification process (VP) involves mixing these wastes after pretreatment with glass frit and other additives in a melter, from which the melt is poured into canisters under carefully controlled conditions to produce the desired glass waste form.

To perform the activities mandated by the Act, several supporting systems will be needed. Two of these systems are the VF and the Canister Transfer System (CTS). The HLW solidification is performed by the VF, and the HLW glass-filled canisters are handled after vitrification by the CTS.

The seismic and structural evaluation of the VF at the WVDP had been an ongoing process between the DOE and the NRC until April 1990. In April 1990, the NRC transmitted a letter requesting several details of component analyses that were not completed by the DOE. The DOE responded in August 1992, outlining the ongoing nature of the work in evaluating the seismic and tornado effects on the VF.

The review presented in this report addresses the seismic and tornado design of the VF, the CTS, and associated equipment and facilities. The sources of information for this evaluation and review are described in Section 4. These documents describe the VF, the CTS, and associated equipment and facilities design, focusing on seismic and tornado effects on the VF. Additional information was obtained during a site visit to the WVDP and from other documents, as referenced.

In the following sections, an overview of VF, CTS, and associated equipment and facilities, configurations, and design procedures is presented.

2 FACILITY DESCRIPTION

This review is expected to be part of the Vitrification SAR, to be issued as a draft by the DOE in May 1994. It focuses only on the seismic and tornado analyses performed for the VF, the CTS, and associated equipment and facilities. An overview of the VF is shown in Figures 2-1 and 2-2. For the purposes of this report, the facility can be considered to be made up of the following systems:

- Primary Passive Containment
- Secondary Active Containment
- In-cell Equipment
- Canister Transfer System
- Miscellaneous Structures
- Equipment Outside Containment Vessel

The major structural system addressed in the analysis is the primary passive containment structure. The primary containment barrier consists of the reinforced concrete vault structure with a stainless steel liner enclosing the Vitrification Cell, crane maintenance room, and heating, ventilation, and air conditioning (HVAC) area. The structure is supported by an integral mat-type foundation embedded in the soil. The soil depth at the site is approximately 70 ft over bedrock. The surrounding sheet metal, steel frame building used for support equipment and valve aisles, is considered expendable under extreme tornado and earthquake conditions. The primary containment barrier has a series of openings and penetrations that are covered by access doors, shield windows, and sealed piping at various locations. A negative internal pressure is provided in the VF cell using a filtered HVAC system to prevent leakage. This system is designed to survive the Design Basis Tornado (DBT) and the Design Basis Earthquake (DBEQ) without interruption.

The other structural systems, as well as the critical equipment, are also addressed in some detail in various documentation that was reviewed, and are identified in Section 4 of this report.

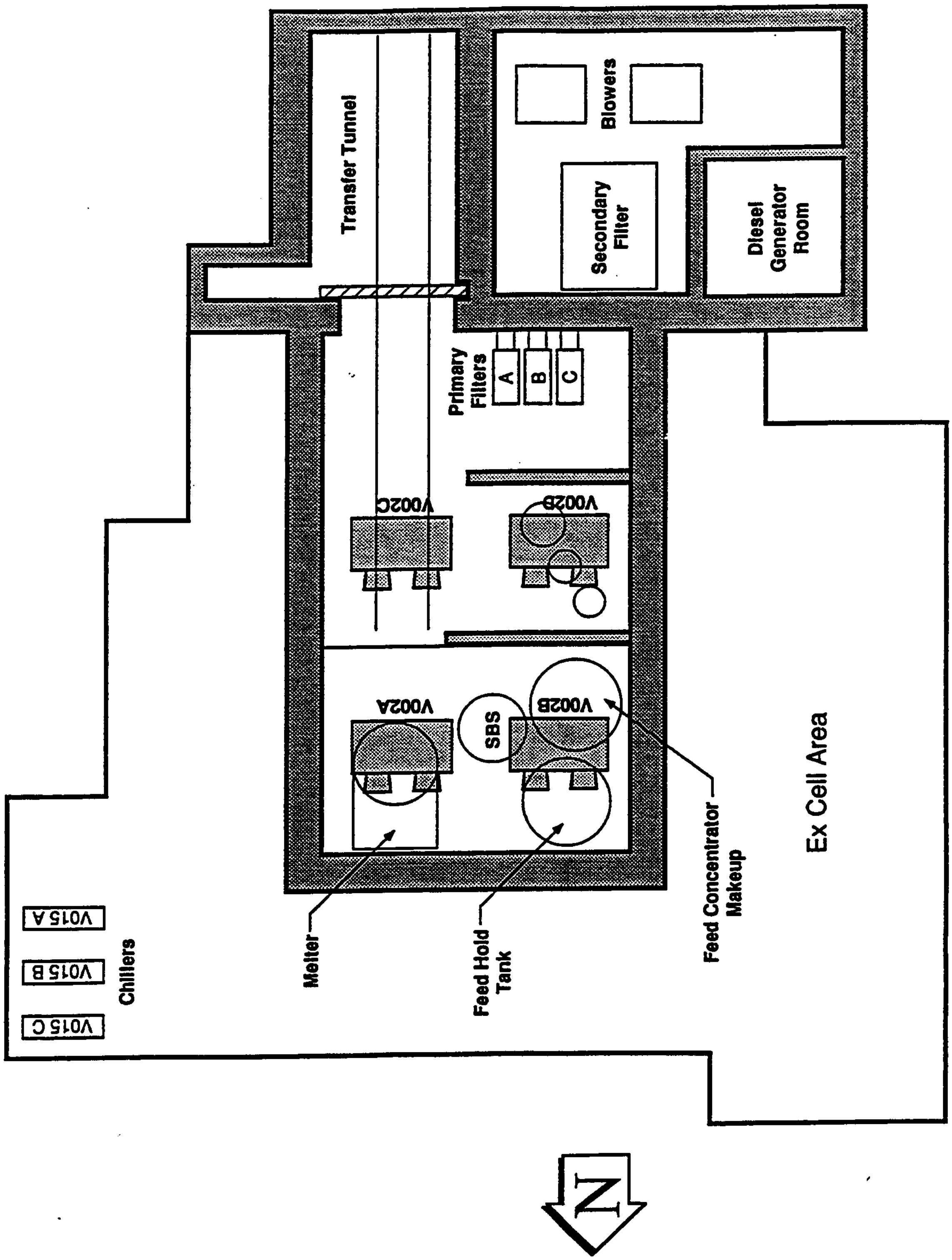


Figure 2-1. Vitrification facility plan (Gates, 1993)

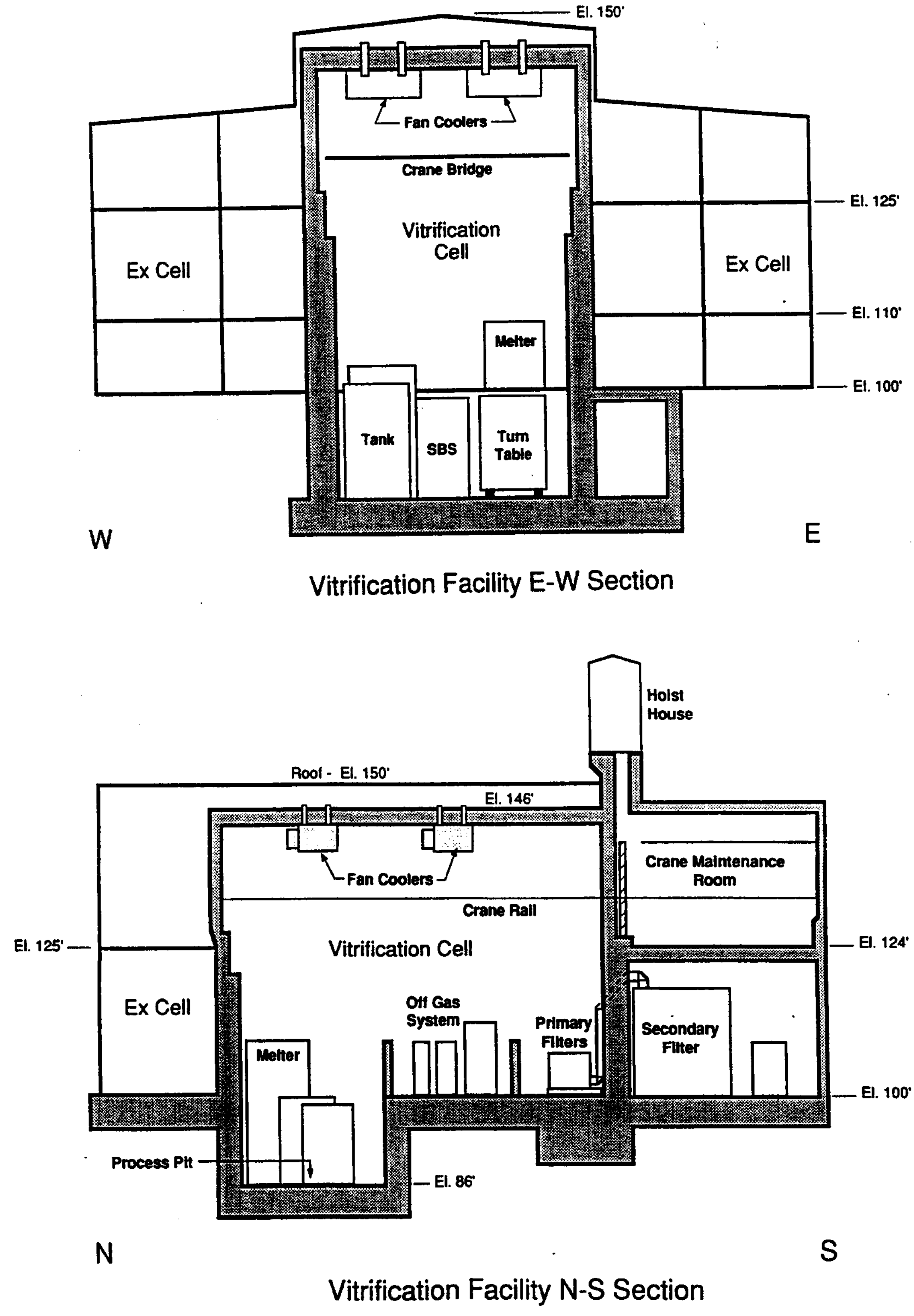


Figure 2-2. Vitrification facility sections, E-W and N-S (Gates, 1993)

3 TECHNICAL APPROACH

The approach used here was to review existing documentation; no original analysis was performed. The review process consisted of reading the documentation, as given in chronological order, followed by a technical review. The review considered the technical aspects of the approaches, procedures, and results given, as well as interpretation of these with respect to historical [American Society of Civil Engineers (ASCE), 1980] and currently acceptable procedures with respect to particular regional needs (Bernreuter et al., 1989).

The areas of review included:

- Identification of the appropriate civil structural elements as well as critical equipment and miscellaneous structures (Table 5-1).

- Proper identification of the design basis loadings: earthquakes, tornado, and wind. The loads are defined in terms of the annual probability of exceedance, peak seismic acceleration or wind speed, and an importance factor [American National Standards Institute (ANSI A58)] for the structure (Figure 3-1). In addition, other loads, such as differential thermal, were considered. Finally, the loads are considered in combination, as given in Figure 3-2.

- Structural analysis procedures used to model the structures, both strength of materials and finite element analysis (FEA) approaches, were reviewed. Of particular concern was the three-dimensional (3D) stick model utilized to approximate the response of the major civil structural elements, including the soil properties. The 3D stick model was an idealization of the physical structure utilizing a series of beam elements (sticks) based on the stiffness and mass characteristics of the structure. This allowed for modeling of impact and responses in two horizontal and one vertical direction. Examples of some of the FEA models used by EBASCO and Dames & Moore (Gates, 1993) are given in Figures 3-3 to 3-6. Each of these will be discussed in some detail in the following sections.

- Procedures to test critical structural elements and equipment were reviewed. This includes both quasi-static testing and seismic qualification.

- Interpretation of results of the analysis and testing directly, and how it relates to the given margins of safety (Figure 3-7). The life of the structure is assumed to be nominally five years.

The following regulations, orders, and criteria related to this program were identified during the review of the documentation available. It should be noted that, of these identified, only the NRC RGs were on hand at the CNWRA for direct review. The others are included by reference.

- West Valley Nuclear Services Co., Inc. (WVNS) WVNS-DC-022 "Design Criteria-Vitrification of High Level Wastes," December 12, 1986.

- WVNS-EQ-264, Rev. 4, "Special Doors for the Vitrification Facility," July 14, 1989.

- West Valley Demonstration Project, Safety Analysis Report (WVDP SAR)

WIND HAZARD SUMMARY

	DOE Moderate Hazard	DOE High Hazard	WVNS SAR
Annual Probability of Exceedance	1×10^{-3} (a)	1×10^{-4} (a)	1×10^{-3} (b)(c)
Fastest Mile Wind Speed (mph)	91 (b)	104 (b)	90 (c)
Importance Factor (I)	1.0 (a)	1.0 (a)	1.07 (d)

Source

(a) UCRL-15910 - Table 5-3, Summary of Minimum Wind Design Criteria
(b) McDonald - Table 8, Wind Hazard Probability
(c) WVNS - WVDP SAR WVNS-SAR-001
(d) ANSI A58.1 - Table 1 and Table 5

TORNADO HAZARD SUMMARY

	DOE Moderate Hazard	DOE High Hazard	WVNS SAR
Annual Probability of Exceedance	2×10^{-5} (a)	2×10^{-5} (a)	1×10^{-6} (b)
Maximum Wind Speed (mph)	50 (b)	50 (b)	160 (c)
Importance Factor (I)	1.0 (a)	1.35 (a)	1.00 (c)

Source

(a) UCRL-15910 - Table 5-3, Summary of Minimum Wind Design Criteria
(b) McDonald - Table 7, Tornado Hazard Probability
(c) WVNS - WVDP SAR WVNS-SAR-001

SEISMIC HAZARD SUMMARY

	DOE Moderate Hazard	DOE High Hazard	WVNS SAR
Annual Probability of Exceedance	1×10^{-3} (a)	2×10^{-4} (a)	6×10^{-4} (b)
Peak Ground Acceleration	0.07g (b)(d)	0.13g (b)	0.10g (c)

Source

(a) UCRL-15910 - Table 4-1
(b) Dames & Moore August 17, 1983 - Seismic Hazard Analysis
(c) WVNS - WVDP SAR WVNS-SAR-001
(d) Minimum of 0.10g required

Figure 3-1. Definition of design basis loading conditions for wind, tornado, and seismic (Gates, 1993)

LOAD COMBINATIONS / ACCEPTANCE CRITERIA

CONCRETE

FOR CONCRETE, **ACI 318** LOAD COMBINATIONS AUGMENTED WITH **ACI 349** LOAD COMBINATIONS FOR TORNADO AND DBE / SSE.

a. NORMAL OPERATING AND CONSTRUCTION LOAD CONDITIONS

1. $U = 1.4D + 1.7L + 1.7H_{Construction}$

2. $U = 1.4D + 1.7L + 1.7H_{Static}$

3. $U = 1.4D + 1.7T_o + 1.4P_o$

4. $U = 1.05D + 1.05T_o + 1.3L + 1.05P_o$

b. SEVERE ENVIRONMENTAL LOAD CONDITIONS

5. $U = 1.05D + 1.3L + 1.3H_{Static} + 1.3W + 1.05T_o + 1.05P_o$

7. $U = 1.4D + 1.7L + 1.7H_{Static} + 1.7W$

8. $U = 0.9D + 1.3W + 0.9T_o + 0.9P_o$

9. $U = 1.05D + 1.3W + 1.3H_{Static} + 1.05T_o + 1.05P_o$

c. EXTREME ENVIRONMENTAL LOAD CONDITION

10. $U = 1.0D + 1.0L + 1.0H_{Static} + 1.0E_{sse} + 1.0H_{Dynamic} + 1.0T_o + 1.0P_o$

11. $U = 1.0D + 1.0L + 1.0H_{Static} + 1.0T_o + 1.0W_t + 1.0P_o$

FOR THE CONCRETE STRUCTURE, U IS THE SECTION STRENGTH REQUIRED TO RESIST THE DESIGN LOADS BASED ON THE STRENGTH DESIGN METHOD DESCRIBED IN **ACI 318-77** CODE.

Figure 3-2a. Definition of load combinations and acceptance criteria (concrete structures) (See page 5-12 for definition of terms.) (Gates, 1993)

LOAD COMBINATIONS / ACCEPTANCE CRITERIA (CONT'D)

STEEL

a. NORMAL OPERATING LOAD CONDITIONS

1. $D + L$
2. $D + L + T_o$

b. SEVERE ENVIRONMENTAL LOAD CONDITIONS

3. $D + L + W$
4. $D + L + W + T_o$
5. $D + L + W + E_{ubc} + T_o$

c. EXTREME ENVIRONMENTAL LOAD CONDITIONS

6. $D + L + E_{sse} + T_o$
7. $D + L + W_t + T_o$

LOAD	COMBINATION	LIMIT
4.2.a	1	S
4.2.a	2	1.5S*
4.2.b	3	1.33S
4.2.b	4 and 5	1.5S*
4.2.c	6 and 7	1.6S*

* Shall not exceed yield strength

"S" IS THE REQUIRED STRENGTH BASED ON ELASTIC DESIGN METHODS AND ALLOWABLE STRESSES DEFINED IN PART I OF **AISC - 1980**. NO INCREASE IN ALLOWABLE STRESSES SHALL BE ALLOWED WHEN DESIGNING CONNECTIONS INCLUDING BASE PLATES AND EMBEDMENTS.

Figure 3-2b. Definition of load combinations and acceptance criteria (steel structures) (See page 5-12 for definition of terms.) (Gates, 1993)

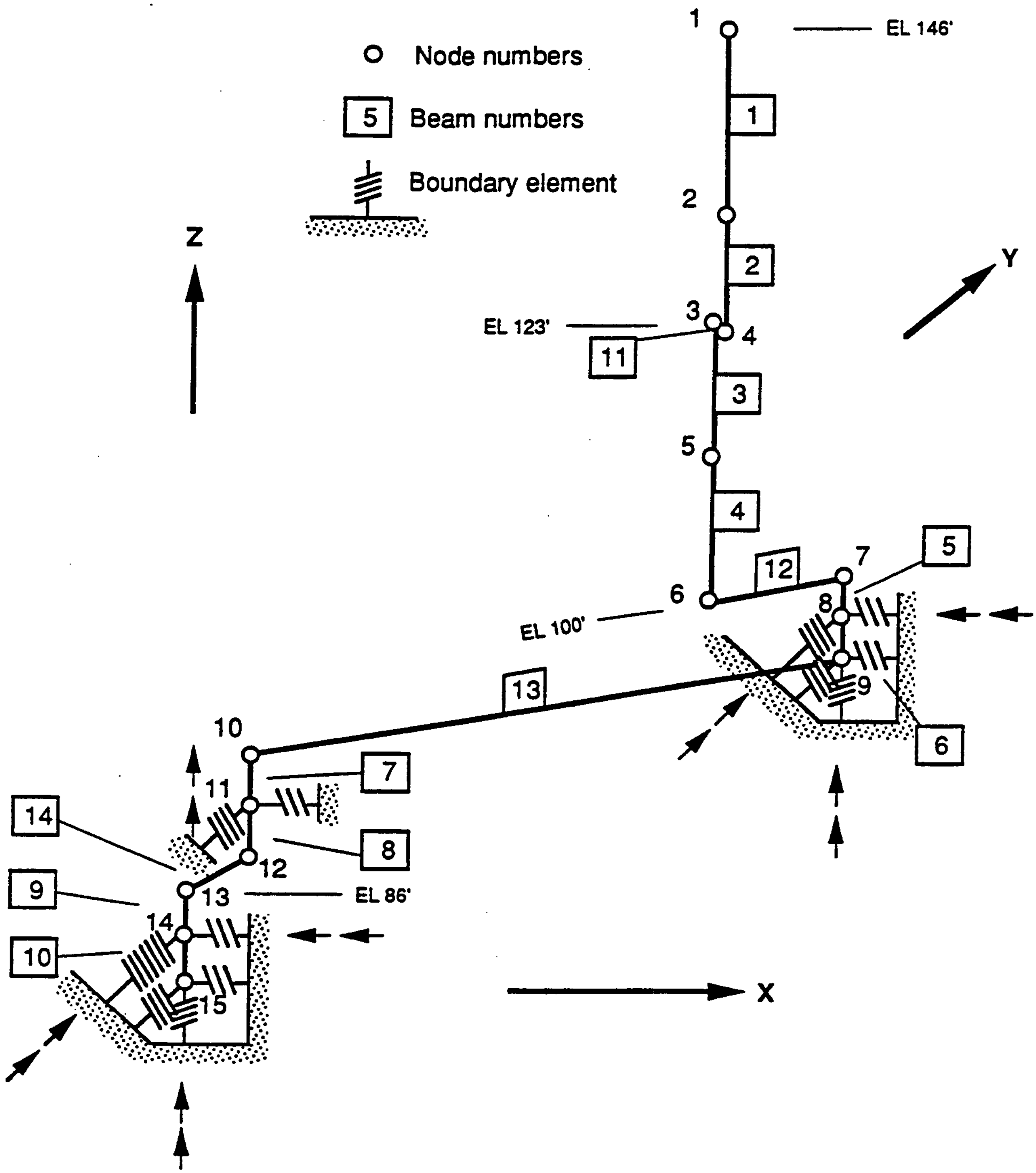


Figure 3-3. VF stick model used by Dames & Moore (Gates, 1993)

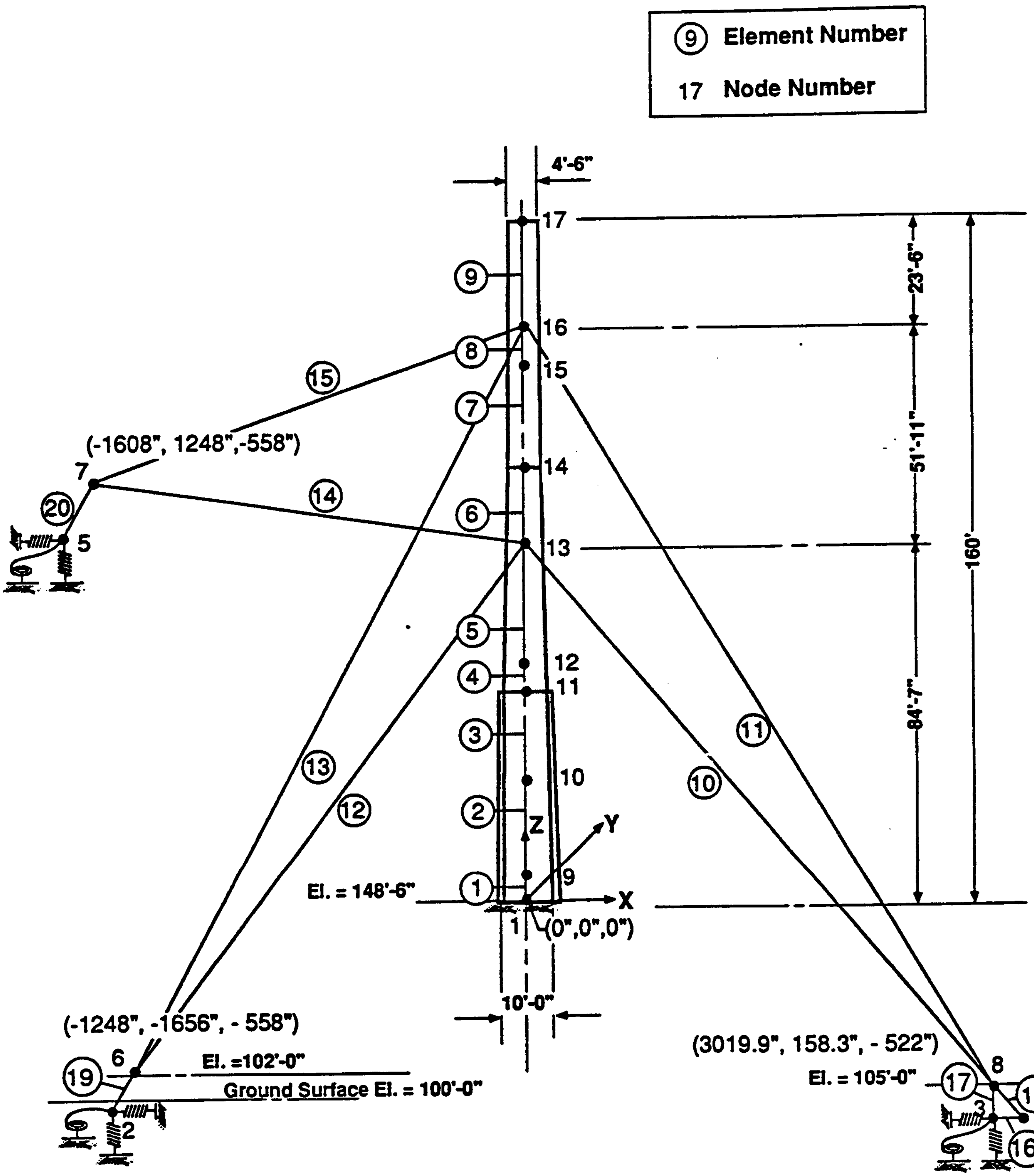


Figure 3-4. Dames & Moore FEA model of reprocessing plant guyed stack (Gates, 1993)

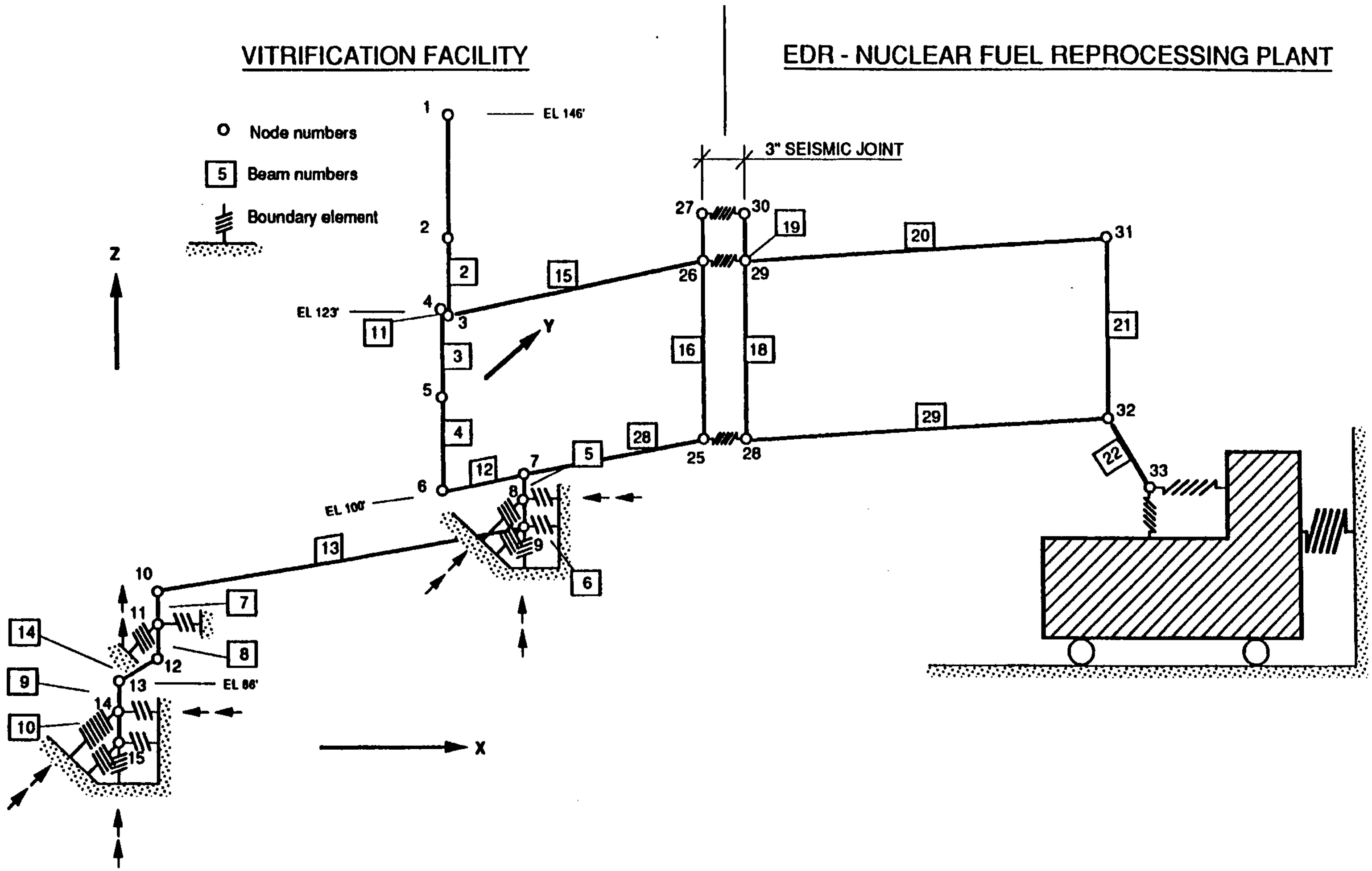


Figure 3-5. Coupled VF/EDR stick model used by Dames & Moore (Gates, 1993)

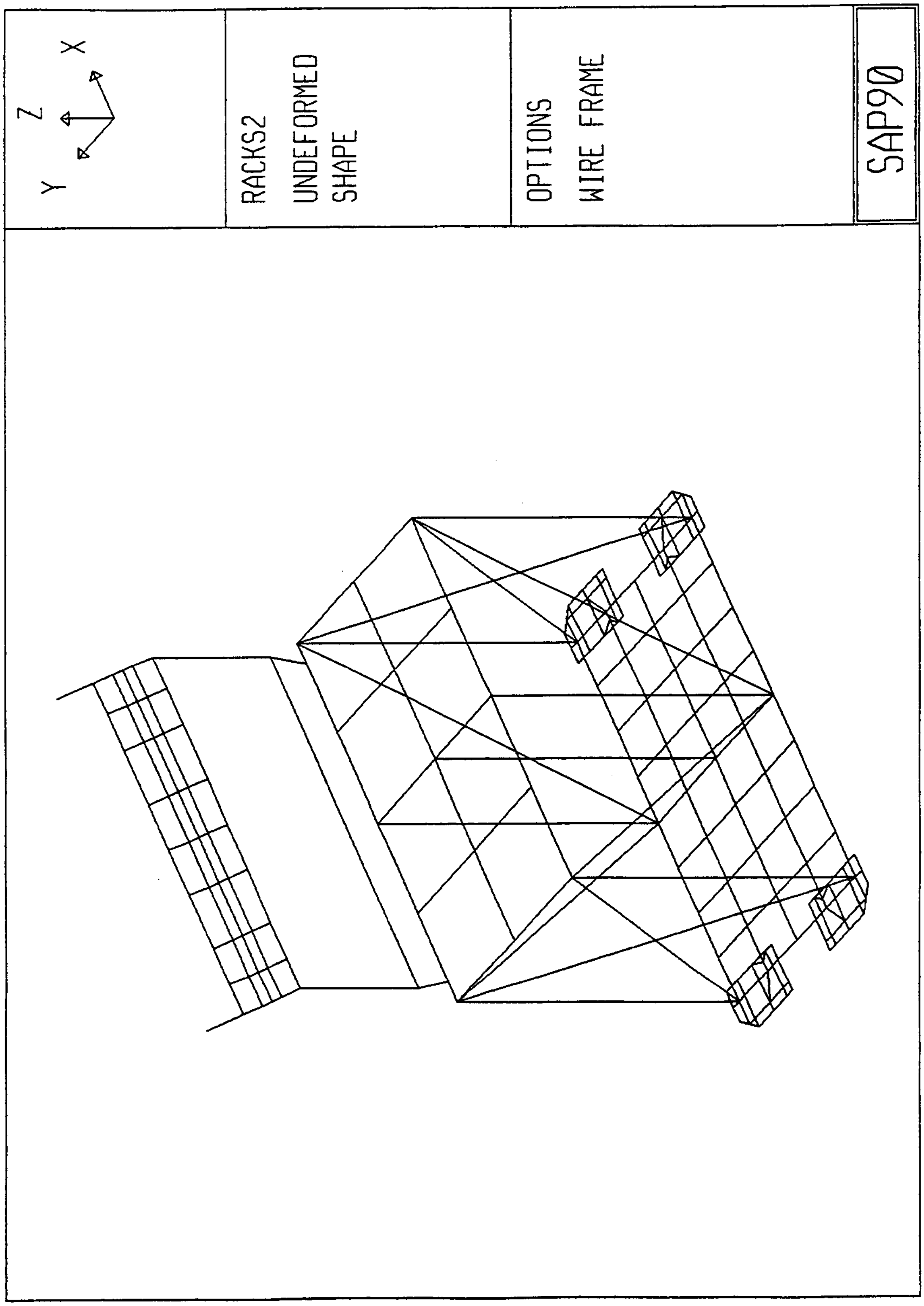


Figure 3-6. FEA of canister storage racks and waste header guide used by Dames & Moore (Gates, 1993)

DEFINITION OF SAFETY MARGIN

$$\text{MARGIN} = \frac{U_{FAIL} - (DL + LL + P_o + T_o + H_{ST} + H_{DY})}{DBE}$$

U_{FAIL} = Failure Strength of Barrier
DL = Dead Load
LL = Live Load
P_o = Operating Pressure
T_o = Operating Temperature
H_{ST} = Hydrostatic Pressure
H_{DY} = Static Soil Pressure
DBE = Design Basis Event, either Earthquake or Tornado

Figure 3-7. Definition of safety margin used by Dames & Moore (Gates, 1993)

- DOE Order 6430.1A, 1989, "General Design Criteria"

- DOE Order 5480.NPH "Natural Phenomena Hazard"

- UCRL-15910, June 1990, "Design and Evaluation Guideline for DOE Facilities."

- NRC, 1973, "Design Response Spectra for Seismic Design of Nuclear Power Plants," Regulatory Guide 1.60.

- NRC, 1973, "Damping Values for the Seismic Design of Nuclear Power Plants," Regulatory Guide 1.61.

4 BACKGROUND DOCUMENTS

To facilitate the review of the analyses for the VF at the WVDP under the DBEQ and DBT, a number of documents and materials were examined. The following is a list of the documents received at the CNWRA for review, together with comments regarding those documents.

- Johnson, N. E., and J. C. Walls, December 1981, "Seismic Resistance Capacity Evaluation of Spent Fuel Storage Racks and Fuel at West Valley, New York," NUREG/CR-2236, by Science Applications, Inc., for NRC, Washington, D.C.

  This is a comprehensive report on the procedures utilized for the seismic analysis of the spent fuel storage racks. The analysis was based on an FEA approach and used time-history procedures for defining the loading and critical responses and stresses. For this analysis, the assumed damping of the spent fuel storage rack system was 10 percent and included material, structural, and added damping due to immersion in the water. The damping value utilized was also related to experimental data for similar structures. This report has been reviewed as an example of a seismic analysis evaluation of facilities at the WVDP.

- Murphy, Donald J., March 18, 1983, "Report on Geotechnical Investigation Proposed Component Test Stand West Valley Demonstration Project, West Valley, New York," by Dames & Moore, for WVNS.

  This was the initial geological study of the site of the VF. The site consists of four strata beneath a 2-ft layer of asphalt and sandy fill. The strata include a layer of dense silty gravel up to 20 ft thick, a layer of clayey glacial till from 20 to 69 ft thick, a layer of dense silty gravel 0 to 5 ft thick, and shale and/or siltstone bedrock at a depth of 70 to 74 ft. The soil was determined to be able to support the average foundation stress of 4000 lbs/ft^2 and the peak stress up to 8000 lbs/ft^2, considering the maximum transient loading.

- Mercurio, William F., April 30, 1985a, "Dewatering Design Report, 8D-1 Tank," by Dames & Moore, for West Valley Nuclear Services Co., Inc.

  This is another geotechnical report that has limited information on the soil characteristics. The objective of the study was to determine the requirements for dewatering of the area surrounding the 8D-1 Tank prior to modifications.

- Mercurio, William F., July 15, 1985b, "STS Building Augered Cast-in-Place Pile Design," by Dames & Moore, for West Valley Nuclear Services Co., Inc.

  This covers the design of the piling system used to support the STS Building. It includes a limited amount of soils data. All the soils data from the studies to this date are consistent.

- Mercurio, William F., May 27, 1986, "Spread Footing Design Parameters for Pump Foundations for Tanks 8D-1 and 8D-2," by Dames & Moore, for West Valley Nuclear Services Co., Inc.

This covers the design of the spread footing used for support of the pumps installed in Tanks 8D-1 and 8D-2. It includes a limited amount of soils data.

- Blickwedehl, Robert R., June 1988, "Report, Subsurface Investigation, Vitrification Facility, West Valley Demonstration Project," by Dames & Moore.

  This document contains comprehensive information of the soil conditions to a depth of approximately 26 ft. The results obtained confirmed the results of the 1983 report at the same site. In addition to the bore logs, information is provided on the results of testing performed on soil samples from the high-level waste transfer trench site. As part of the soil-structure interaction evaluation, field testing by the downhole seismic testing method was performed, in order to obtain the required elastic properties as in-situ values measured at low strain levels, <0.00001 percent. In addition, resonant column (RC) tests were performed on soil samples to determine the dynamic properties in the nominal strain range of 0.0001 to 0.01 percent and cyclic triaxial tests in the nominal shear strain range of 0.1 to 1 percent. As expected, the basic soil shear modulus decreased with increasing strain while the damping increased with increasing strain.

- Maestas, E., Letter of Transmittal to Davis Hurt, NRC, dated September 8, 1989. DOE Idaho Operations Office, West Valley Project Office.

  This letter transmits the three following progress reports produced by Dames & Moore, each dated August 11, 1989, to the NRC. These are reviewed, but, since they have been superseded by more recent documents, only limited discussion is given.

- Gates, William E., "Progress Report, Confinement Barrier Integrity Review of Shield Doors, Vitrification Facility, West Valley Nuclear Services, Inc.," August 11, 1989a, Dames & Moore.

  This document cites two problems identified by Dames & Moore: support for gravity loads for Door 1 and tornado missile rebound for Doors 3, 4, 5, and 7. The second problem was purportedly mitigated by a design change for the latch pins for Doors 3, 4, and 5, but a need for further review or mitigative action was proposed for Door 7. Table A of the report has incomplete data on doors for certain combinations of door component and/or failure mode. The credibility of the various failure mode combinations remains to be investigated. Also, Table A indicates areas that require further review to evaluate the consequence of potential failures for Doors 4, 5, and 7. At this time, the resolution of a gravity problem with Door 1 was pending and was considered to be an item for future followup with the DOE. (Note that these issues are addressed in Gates, 1992).

- Gates, W. E., A. R. Porush, and R. W. Kupp, "Tornado-Driven Missile Penetration Resistance of Shield Windows on Vitrification Cell for West Valley Nuclear Services, Inc.," August 11, 1989, Dames & Moore.

  The shield windows are considered a weak element in the primary passive containment system when subjected to impact by a tornado-driven missile. The basic geometry of the window is shown in Figure 4-1. It consists of multiple layers of leaded glass supported in

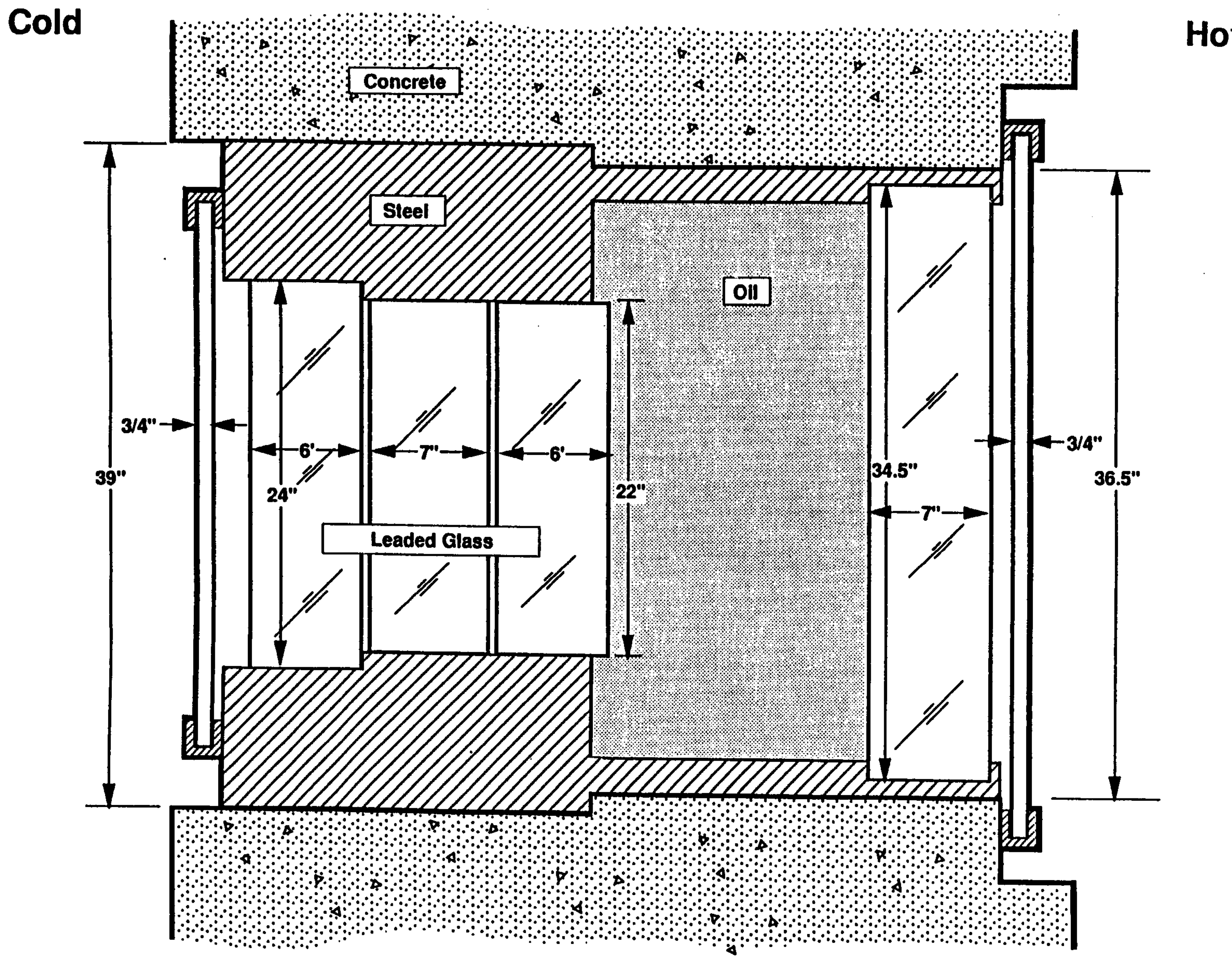


Figure 4-1. Geometry of the shield windows (Gates, 1993)

a steel frame. Several thinner sheets of glass are provided on the cold and hot sides of the windows for protection of the leaded glass.

Analytical or test data on the penetration resistance for windows of this type is not given in the reports reviewed. Based on a lack of such information in the industry, it was assumed that the glass would be punctured by the missiles and the leakage rate would be determined. The size and resulting speed of the tornado-driven missile used in this program is consistent with established procedures.

A worst-case scenario presented by Dames & Moore (Gates et al., 1989) was used to determine possible effects of a window being completely blown through its frame by a tornado-borne missile. This scenario led to the conclusion that "the post-tornado winds would produce sufficient negative pressure that hazardous quantities of gaseous materials could be released from the cell in an unacceptable radiological accident." The size of the penetration used in the analysis was next reduced, and the resultant loss of 37 cubic ft of radioactive gaseous materials was considered acceptable. Reducing the size of the penetration used in the analysis was justified by concluding that much of the glass rubblized during missile penetration would remain in place afterwards and thus act to restrict air flow. This conclusion was based on "simple momentum considerations between the design basis missile and the physical geometry of the multiple layered windows." The negative pressure created by the HVAC system, designed to withstand tornado loads and remain functional, is reported to maintain radiological consequences within acceptable limits.

It was stated that "it is the opinion of experts in glass manufacture and missile testing of glass that the DBT for the WV site will not penetrate the multiple layers of heavy glass that form the shield windows on the Vitrification Cell." Recent unpublished work at Southwest Research Institute (SwRI) showed that an eleven pound projectile with a rounded tip, traveling at over 500 ft/sec, penetrated a 4-in.-thick glass, with approximately 100 ft/sec residual velocity (SwRI, 1992). Based on estimates of the loss of kinetic energy for this projectile, it was estimated that a 6-in.-thick glass would have stopped the projectile, although no estimation of through-pane cracking was made. Based on a relationship of the kinetic energy, proportional to (mass) $\times$ (velocity)^2, of this light but high-velocity projectile and the heavy but low-velocity pipe projectile, the likelihood of penetration of a single pane of thick glass is small, much less penetration of the entire assembly. Based on the SwRI tests, this item is considered closed. The window will not fail under the tornado missile load, allowing no radiological release to the atmosphere.

- Gates, William E., "Progress Report, Primary Confinement Barrier Integrity Review, Vitrification Facility, West Valley Nuclear Services, Inc.," August 11, 1989b, Dames & Moore.

The scope of this review by Dames & Moore was the seismic resistance of primary confinement civil structural elements of the VF cell (excluding the shield windows and shield doors). The majority of the information presented in this document is repeated in a number of later documents. The original results are based on using a 3D stick model similar to that shown in Figure 3-3. No details of the geometry or beam properties were given, so it was not possible to verify them. The only interpretation is based on looking at the dynamic responses based on past experience with similar structures.

The damping values used in the original 3D stick model were 7 percent for the structural elements and 20 percent for the soil to account for both material and radiational damping. The review process used by Dames & Moore utilized a composite modal damping of 15 percent based on the relative strain energy in the elements. Therefore, it is evident that the majority of strain is assumed to be in the soil. The value of damping for the structural elements used is consistent with the damping values given in RG 1.61 for reinforced concrete at the Design Basis Event (DBE) level. The damping in the soil is a function of both the internal damping and radiation (geometric) damping, $D = D_{int} + D_{rad}$ (Das, 1983). The radiation damping is to account for loss of energy as waves propagate from the foundation into the surrounding soil.

Accurate determination of the damping characteristics of soils for embedded foundations under dynamic loadings is difficult. To accurately define the strain field in the soil surrounding the foundation, it is necessary to develop a full 3D solids model of the system. In this way, strain levels at different points on the structure due to sliding, vertical motion, rocking, and torsion of the foundation can be determined. Because material properties of the soils are highly strain-dependent, it is necessary to perform nonlinear time-history solutions to obtain accurate results. From discussions (Gates, 1993), it appears that this type of analysis is currently in progress, but results have not been presented. Results of this type of model can be used for subsequent simplified analysis.

The alternative procedure used in the original analysis, and that performed by Dames & Moore (Gates, 1989b), is to idealize the soils utilizing boundary elements consisting of springs and dampers. For the 3D stick model (Figure 3-3), boundary elements were used at three different physical locations. At two of the locations, boundary elements are placed at different elevations to account for embedment of the foundation. The source of the values used in determination of the stiffness and damping values used in this model are not given. It appears that they are based on the simplified approaches similar to those given by Das (1983), Novak (1974), and Veletsos and Meek (1974). From the discussions given in the various documents, it is not apparent whether a linear or nonlinear boundary element was utilized.

The specified DBEQ of 0.1 g peak acceleration is likely to produce overall strain levels in the soil that are low when compared to the larger earthquake events representative of the west coast region. Based on the tabulated values of displacement and overall foundation size and soil depth as the reference length, the maximum estimated strain is 0.04 percent. This would indicate the use of damping values from 5 to 10 percent rather than the 20 percent used. Assuming a linear response, the decrease in damping would result in increased strain, up to double, representing a mean damping value of 15 percent. The estimates given here are very simplistic and more detailed analysis is required to define the appropriate levels of damping used, since this is critical in the calculations.

As a conservative approach, it is recommended at this time that no more than 15 percent damping be utilized for the soil elements. In addition, a conservative approach would be to utilize a structural damping value of 4 percent based on the operating bases earthquake (OBE) level for reinforced concrete structures (RG 1.61). The strains in the concrete as a result of the 0.1 g earthquake are likely to be more closely related to the OBE responses given in RG 1.61. In both cases, a 3D FEA stick model was used to represent the VF cell

(Figure 3-3). The results presented are in terms of modal frequencies, peak displacements at various locations, elevated response spectra, and the forces and stress in the elements. The response spectra indicate a primary response of 5 to 7 Hz, corresponding to the first natural frequency of the VF cell. Under earthquake loading, the margins of safety for structural yield or failure range from 3 to 6, except for the roof hatch, for which the margin of safety was 1.5. Reduction of the soil damping from 20 percent to 10 or 15 percent could lower these by as much as a factor of 2. This could result in margins of safety less than 1.0. Therefore, resolution of the soil damping values is critical for resolution of this open issue.

Table 9 of the report indicates that cracking of the roof slab and the top portion of the walls of the VF could occur during an earthquake of magnitude less than the DBE, when out-of-plane bending is combined with thermal stresses. However, the report states that, for ultimate collapse, the structure "probably has a margin of safety on the order of 5 or 6 times the DBE, based on engineering judgement and experience." The assumption made is that cracking is not considered a failure because there will be no potential for release of radioactive material as a result of cracking. Based on the thickness of the structure and formation of cracks on the tension side only, this is an appropriate assumption.

In addition to structural failure, the potential for contact between the VF cell and the Equipment Decontamination Room (EDR) (Figure 4-2) of the existing Nuclear Fuel Reprocessing Plant is determined to be remote. The maximum closure of the existing 3-in. gap is reported to be 0.8 in. Reduction of the soil damping to 10 percent could increase the deflection by a factor of 2 with a corresponding reduction in the margin of safety. The model utilized for this analysis is given in Figure 3-5. It basically consists of the VF stick model coupled to another stick model of the EDR.

The report also states that "further study should be conducted on the influence of radiation, chemical attack, freezing and thawing, etc., on the physical properties of the RodoFoam" which comprises the coupling between the VF and the EDR. Although coupling through the RodoFoam is described as "insignificant," this could change if the RodoFoam material properties stiffen with age or environmental exposure to the extent that coupled earthquake response of the VF and EDR is significantly affected. A description should be given of the program planned to pursue the "further study," or bounding calculations should be provided to show that such a study is not necessary.

- DesCamp, V., W. E. Gates and R. W. Kupp. November 7, 1989. "Synopsis and Presentation Materials for Meeting on NRC Progress Review Vitrification Facility Confinement Barrier and Margin Assessment at NRC Headquarters," Rockville, Maryland.

  The synopsis of the meeting included the following:

  Based on the presented data, the NRC stated that their report will indicate that the design of the civil structure and its seismic analysis is adequate. The major open item was that WVDP was to provide some justification of the soil damping values. Work on this is currently in progress. Another action item for WVDP was to provide a description of the scope, parameters, methods, and results of the thermal analysis of the civil structure. This

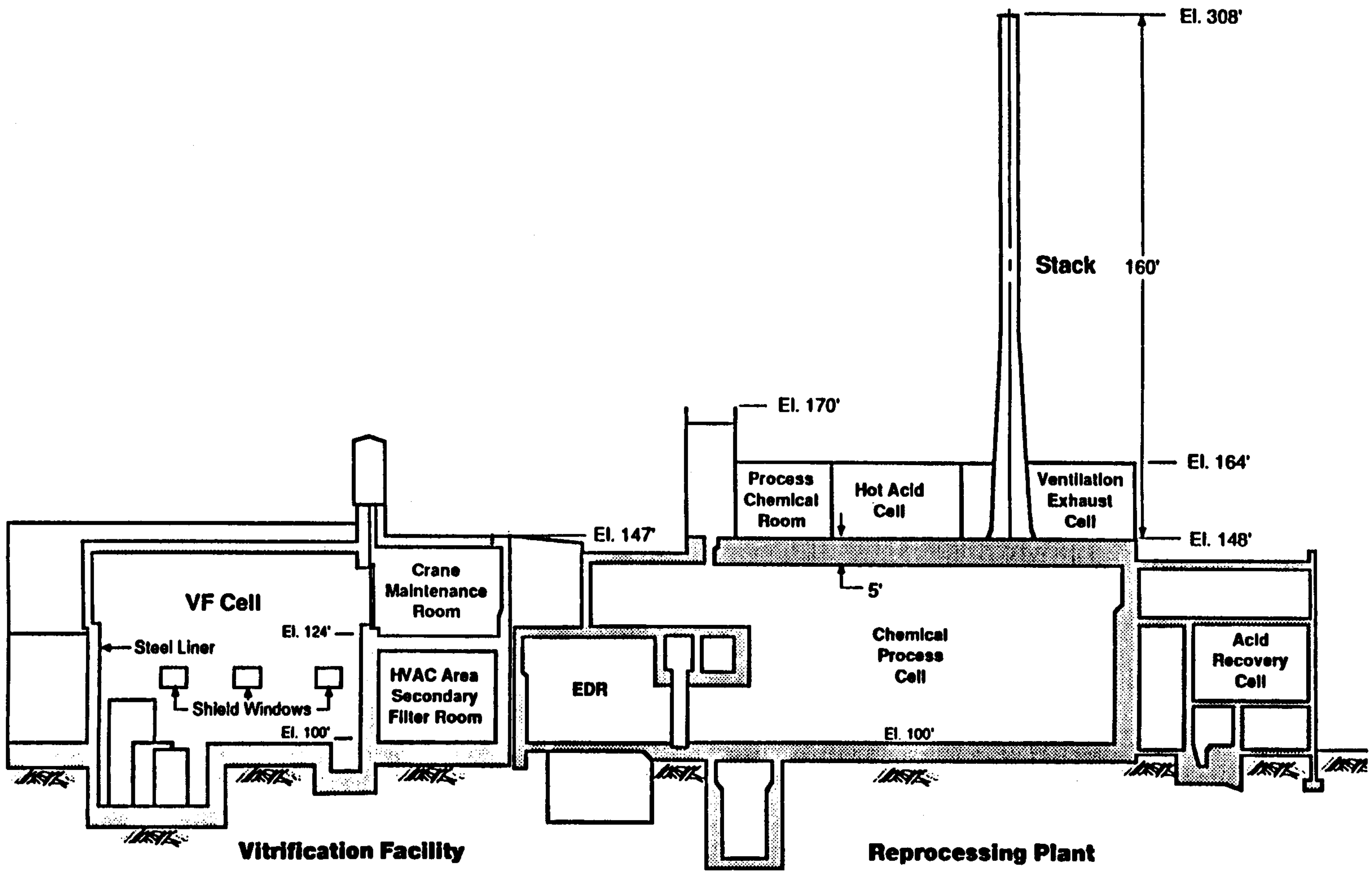


Figure 4-2. Reprocessing plant and VF elevation (N-S) (Gates, 1993)

was presented in terms of a letter indicating the simplified strength of materials approach used.

The NRC indicated that the approach taken for the determination of the consequences of a penetration of the shield window was acceptable but that the Design Criteria (DC-022) had to be modified.

The open items at that time included:

(i) Finalize the door report by Dames & Moore (closure reached in Gates, 1992).

(ii) Completion of the review of the main plant stack by Dames & Moore (closure reached in Dames & Moore, 1992).

(iii) Prepare Safety Evaluation Report on Off-Gas Treatment by NRC (open).

- Gates, William E., "Design Review of Vitrification Facility Confinement Barriers," November 7, 1989c, Dames & Moore.

This is basically a repeat of the information in the Gates (1989b) report. The major additional information was in the area of the analyses procedures for determining the levels of release of radioactive material to the environment as a result of penetration of the cell windows. The open items brought up by this presentation included:

(i) Thermal reinforcing steel in VF cell walls and roof (closure reached in Gates, 1993).

(ii) Process Plant stack under earthquake and tornado loading (closure reached in Dames & Moore, 1992).

(iii) Impact of Canister Cart on the Primary Filters (closure reached in Gates, 1993).

- Price, J. D., Memorandum of November 30, 1989, to R. Davis Hurt of NRC, Subject: Status of the West Valley Vitrification Facility Seismic/Tornado Review, Science Applications, Inc.

This is a summary of all the issues given in the previous reports. No new information was presented.

- Hurt, R. D., Letter of April 2, 1990, to W. Bixby of West Valley Project Office U.S. DOE, NRC Comments on the WVDP Seismic and Tornado Analysis for the Vitrification Facility, Washington, D.C., NRC.

The comments provided to the NRC (Price, 1989) are adapted into a letter to the DOE (Hurt, 1990).

- Rowland, T. J. Letter of August 7, 1992, to G. Comfort of NRC, Responses to U.S. NRC Comments on the WVDP Seismic and Tornado Analysis for the Vitrification Facility, West Valley, New York, U.S. DOE.

  This letter, with its enclosure and exhibits, represents the latest review correspondence between the DOE and NRC concerning outstanding items of discussion concerning seismic analyses for the VF at the WVDP. This letter indicated that the final report on the seismic design for the High-Level Waste Transfer System (HLWTS), prepared by EBASCO and reviewed by Dames & Moore, would not be available until the second quarter of 1993. In addition, the CTS design and seismic analysis was under review. For all in-cell equipment that requires seismic support or is required to operated during and after a seismic event, seismic analysis and/or testing will be performed. The seismic analysis overview on all of this equipment is in differing stages of completion and has not been put in final form.

  A presentation of the simplified thermal analysis of the concrete walls indicates an acceptable margin of safety for this loading condition. This appears to close out this issue.

- Rowland, T. J. Letter of Transmittal October 5, 1992 to G. Comfort of NRC, Responses to U.S. NRC Comments on the WVDP Seismic and Tornado Analysis for the Vitrification Facility, West Valley, New York, U.S. DOE.

  This letter transmits three progress reports discussed below. One report "Progress Review Report" by Dames & Moore was scheduled for completion on October 30,1992, and was to be forwarded at that time. This document has not been made available to the CNWRA as of this time.

- Gates, William E., August 12, 1992, "Confinement Barrier Integrity Review of Special Doors — Vitrification Facility — West Valley Demonstration Project," Dames & Moore.

  This report summarizes the structural design review and integrity assessment of the eight special doors of the VF that function as critical elements of the primary confinement barrier system. The special doors on the exterior of the facility need to stop the tornado missile and remain in place. Operability and/or leak prevention is not a requirement for the special doors.

  The basic DBEs include:

  Wind loads (W)
  Tornado loads (W_t) including wind, differential pressure, and missiles
  Seismic loads (E)
  Dead loads (D)
  Thermal loads (T_0)
  Pressure loads (P_0)
  and appropriate combinations for normal operating and severe and extreme environmental load conditions.

Three separate methods of investigation were used by Dames & Moore (Gates, 1992) to assess the structural integrity of the shield doors under extreme environmental conditions.

These included: (i) analytical methods, (ii) experimental methods, and (iii) engineering judgement. The comprehensive list of failure modes for the doors included:

— Yielding of the door leaf in bending, allowing breaching of the door jambs.

— Penetration of the doors by the tornado missile.

— Failure of the door head and sill stops through plastic hinging or shear failure, allowing breaching of the door jambs.

— Failure of the latch pins, resulting in unrestrained opening of the door.

— Failure of the latch pin assembly, resulting in unrestrained opening of the door.

— Shear failure of the door hinges, permitting the doors to drop and fall open.

— Shear failure of the door head, sill, and jamb embedments in the reinforced concrete wall, permitting the whole door support frame to fall out of the concrete wall.

This list is complete and represents the most likely failure modes.

Table 4-1 of Gates (1992) summarizes the results of the analysis in terms of the margin of safety for the failure modes given above. The combined earthquake and dead load demand on the doors and their support hardware was based on using the peak response for the response spectra provided in WVNS-EQ-264. This document was not available at the CNWRA, and it is assumed that these response spectra are based on elevated response spectra at the critical locations. Since these spectra are most likely based on the use of 20 percent damping in the soil, the loading could be increased by a factor of up to 2 if a lower damping value is used based on the anticipated strain levels in the soil. This increased loading could result in unacceptable margins of safety, based on the conservative analysis performed. It may be necessary to perform more complete dynamic analyses of the door systems to get a true indication of their strength characteristics.

The methodology for tornado missile load analysis on the special doors is based on a two step approach. Based on the ratio of the duration of the impact pulse (t_d) to the period corresponding to the natural frequency (T) of the system, the response is assumed to be either an equivalent static, $t_d/T < 0.1$, or dynamic, $t_d/T > 0.1$, response. This is common engineering practice. Based on discussions in the text of this report, all analyses were done in terms of the equivalent static analysis, although no supporting calculations are shown. It appears that these are contained in the August 12, 1992, report, "Structural Design and Review Calculations, Special Doors," produced by Dames & Moore, that was not available at CNWRA. Based on the results presented, all the issues outlined in the August 11, 1989, Dames & Moore documentation associated with the special doors have been resolved and the designs have adequate margins of safety.

- September 10, 1992, "Wind, Tornado and Seismic Vulnerability Analysis for Reprocessing Plant Guyed Stack - West Valley Demonstration Project," Dames & Moore.

This report summarizes the structural behavior of the existing 160-ft Reprocessing Plant ventilation stack (Figure 4-2) under extreme environmental loads that include wind, tornado and earthquake. The intent of the study was to realistically assess the collapse potential of the stack and the risk that such collapse might pose to the VF.

In the modeling of the stack, it was assumed that the base was fully restrained. For a slender system of this type, it is difficult to obtain a moment restraint at the base, even with the expanded diameter and the addition of gunite to the lower portion of the stack. Realistically, the first mode of the system will be below the calculated 2.34 Hz when the base compliance is taken into account. The best way to verify the model is to take field measurements under ambient vibrations to determine the lowest modes. For this analysis, a 3D FEA beam model was utilized (Figure 3-4).

The response spectra used to define the input into the stack was based on an earlier analysis of the Reprocessing Plant and is dominated by the resonant response of the stack. By matching the calculated first mode of the stack to the stack base response spectra, the current model of the stack is forced to correspond in frequency to the earlier model. The net effect is to produce a conservative analysis.

Collapse of the stack is based on three assumed failure modes:

(i) Moment capacity of the stack is exceeded
(ii) Anchorage capacity of the base of the stack is exceeded
(iii) Failure of the guy system

representing the most credible failures based on both the geometry of the structure and the loading.

The worst margin of safety, 1.2, is associated with a bending failure in the stack just above the gunite sheath, under the DBT. Even if this failure were to occur, the possibility of any damage to the VF is remote. Although the resulting margins of safety are given, no calculations are shown supporting these. It is assumed that these calculations are contained in the structural calculations report produced by Dames & Moore dated February 17, 1992 (not available for review).

- Gates, William E. and R. M. Semple, August 24, 1992, "Geotechnical Investigation of High-Level Waste Transfer System — West Valley Demonstration Project," Dames & Moore.

This report presents the results of a geotechnical investigation to assess the dynamic soil properties for the HLWTS. Of primary interest to this analysis are the recommended G/G_{max} curves for the clay fill (Plate 19), clay till (Plate 20), and lacustrine clay (Plate 21), as well as the recommended damping ratio for cohesive soils (Plate 22) (Figures 4-3 to 4-6). This information should be utilized as a basis for the analysis of the VF to determine the level of radiational damping present in the soil. The issues are the strain- dependent nature of the soil properties and determination of the composite strain levels surrounding the foundation. This is necessary for verification of the simplified soil stiffness and damping characteristics used in the 3D stick models.

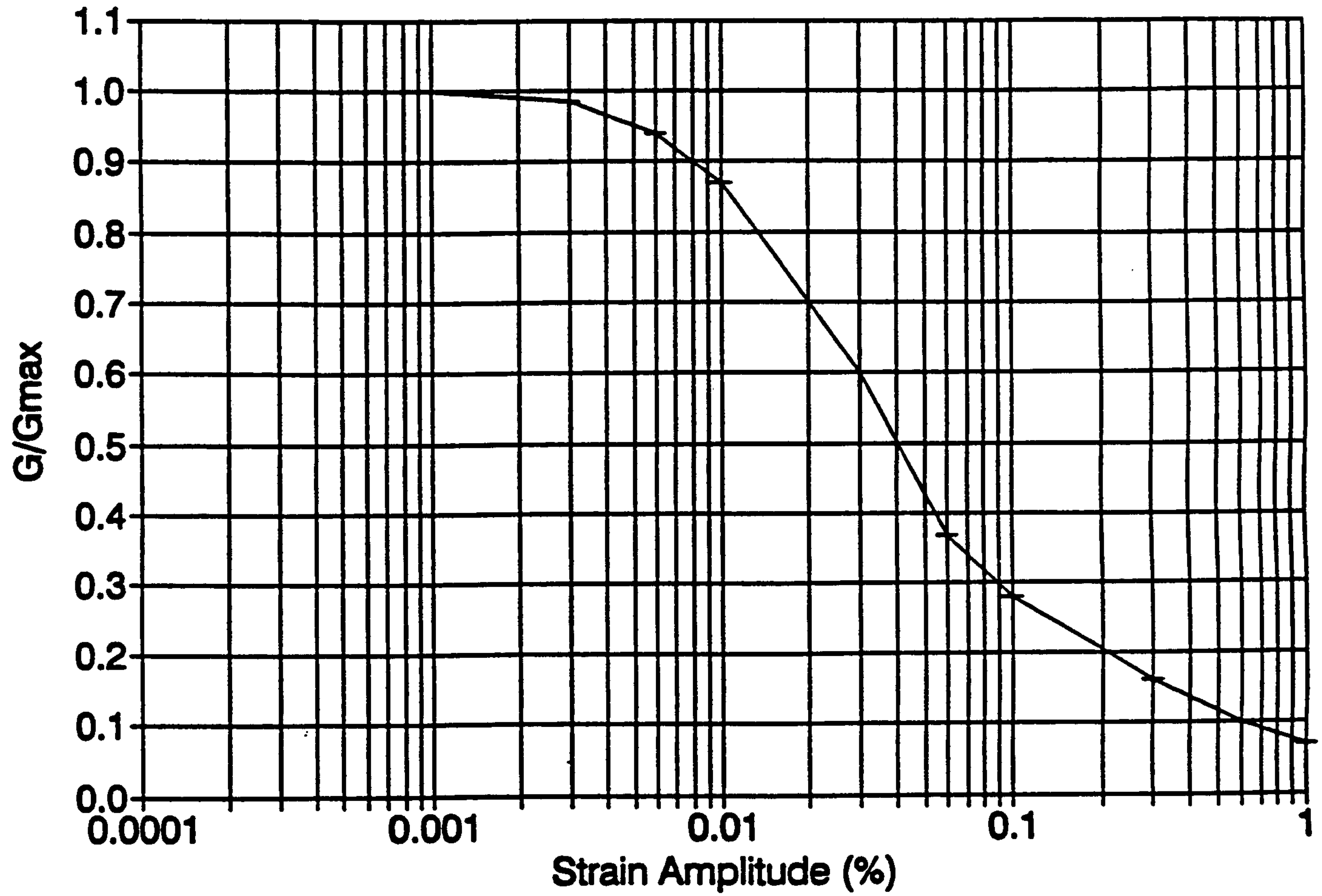


Figure 4-3. Recommended G/G_{max} for clay fill (Gates and Semple, 1992)

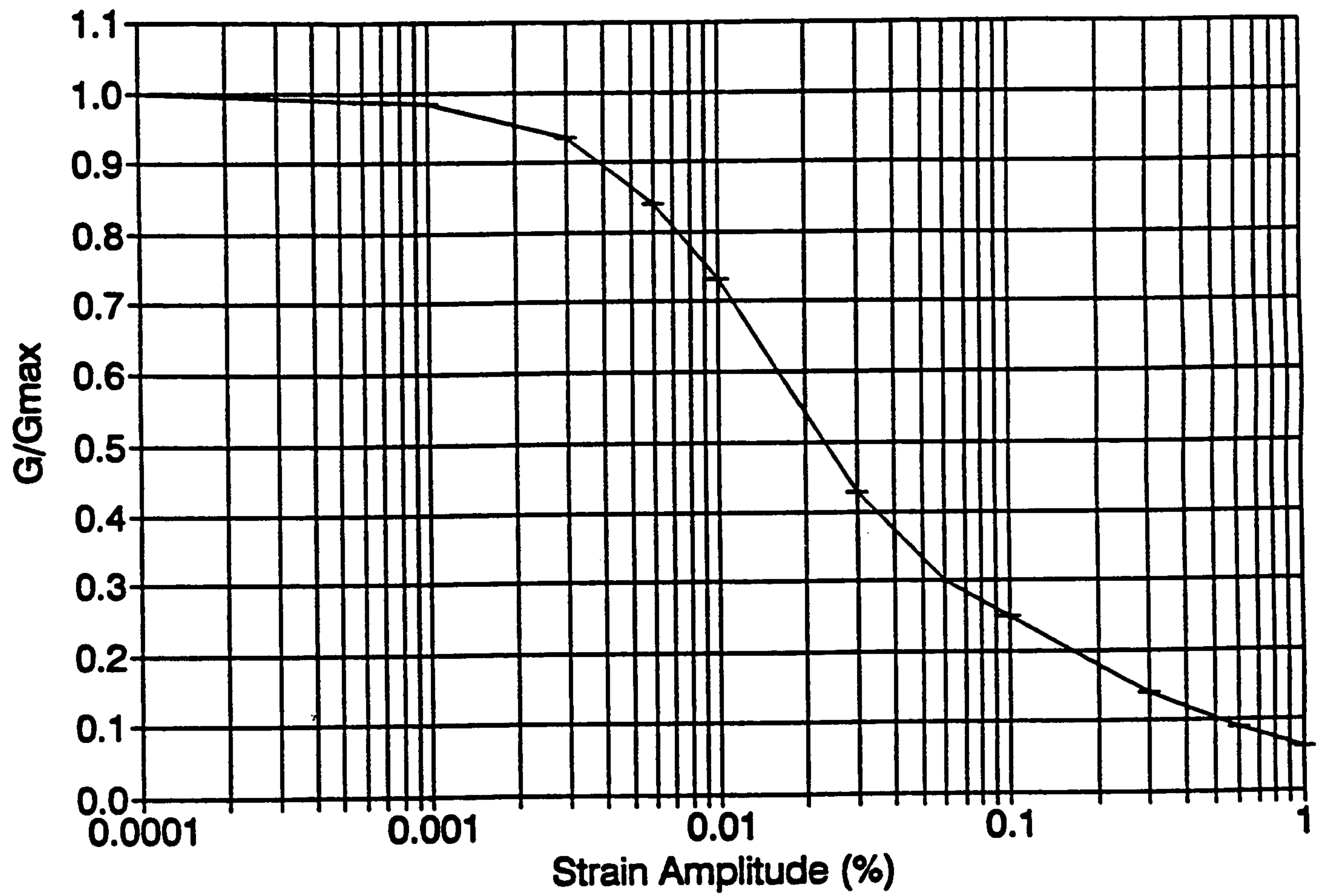


Figure 4-4. Recommended G/G_{max} for clay till (Gates and Semple, 1992)

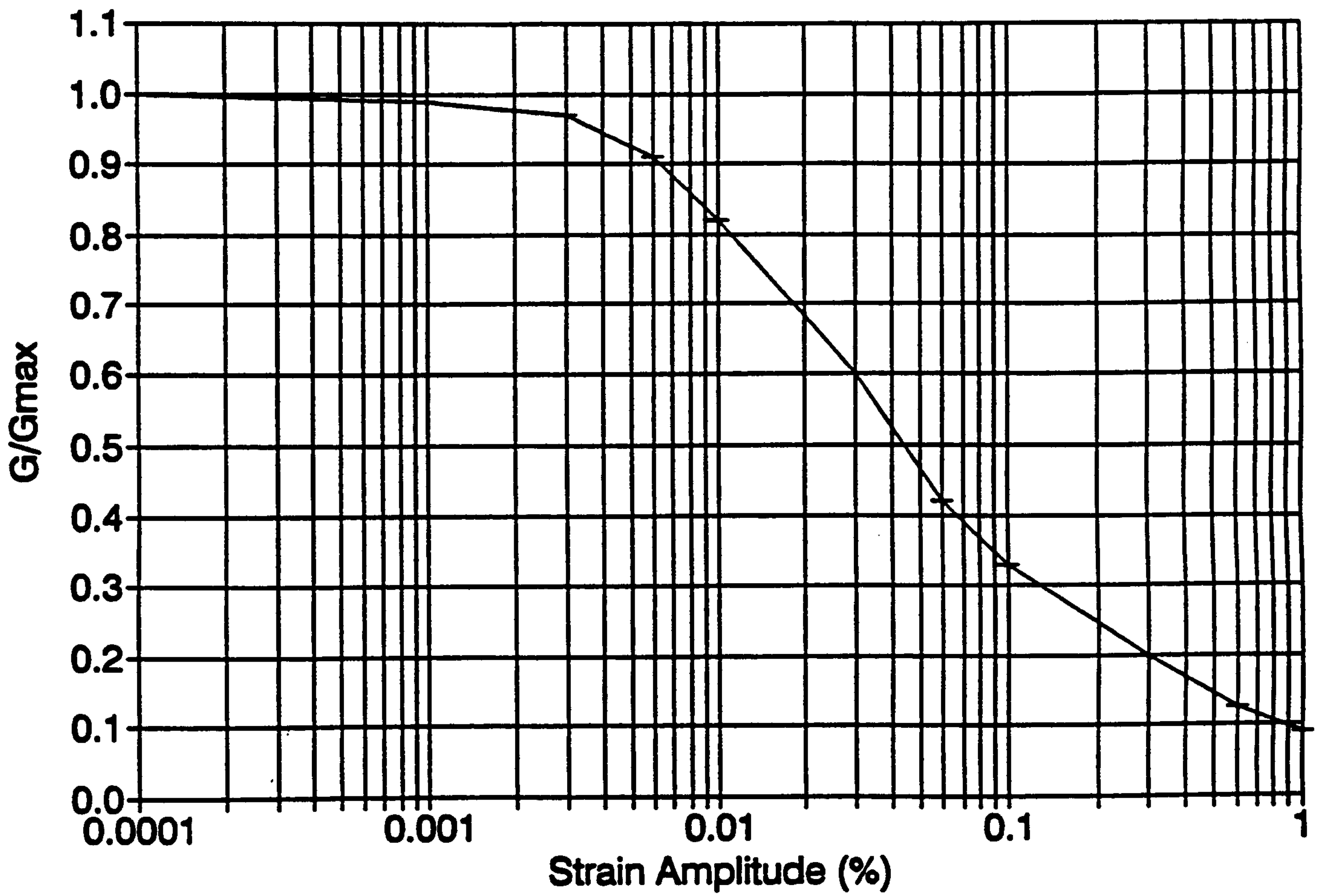


Figure 4-5. Recommended G/G_{max} for lacustrine clay (Gates and Semple, 1992)

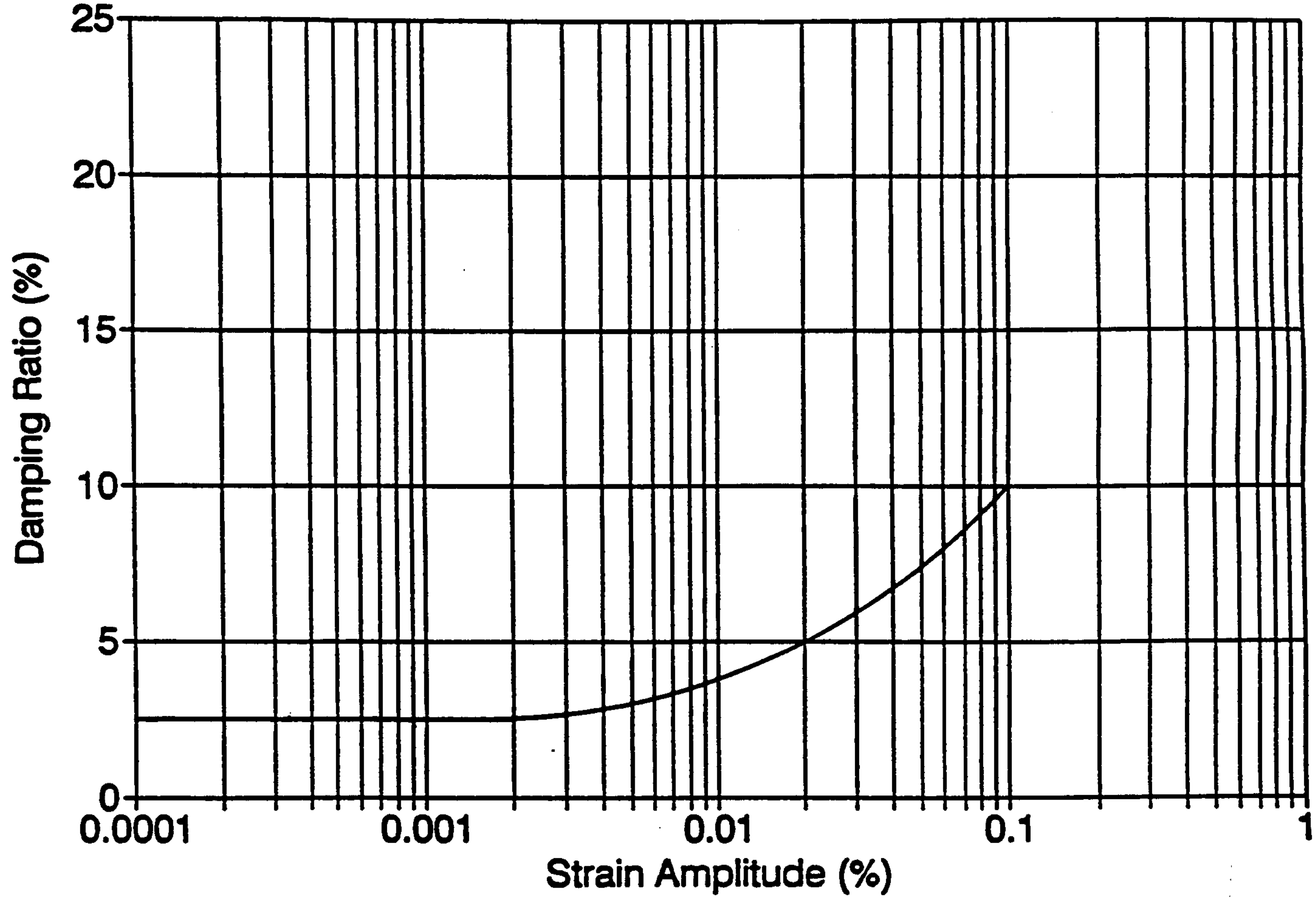


Figure 4-6. Recommended damping ratio for cohesive soils (Gates and Semple, 1992)

The final set of documentation reviewed consisted of the presentations made during a site visit on February 23 and 24, 1993.

- DesCamp, Victor A., February 23, 1993, "Design Strategy and Design Criteria, West Valley Nuclear Services Co., Inc.," Viewgraphs.

- DesCamp, Victor A., February 23, 1993, "Vitrification and Site Design Engineering, West Valley Nuclear Services Co., Inc.," Viewgraphs.

- Gates, Bill, February 23, 1993, "Confinement Barriers, Design Review and Margins Assessment for Vitrification Facility, Dames & Moore," Viewgraphs.

This was a comprehensive presentation on the design review and margins assessment for the VF. This included both a discussion of the design review procedures, and responses to some earlier questions posed by the NRC. An overview of the VF systems that were considered included the primary passive confinement, secondary active confinement, in-cell equipment, and canister transfer system. The individual components in each were then classified in terms of the safety and service classes and the resulting quality level. Results of the design review and margins assessment for some of the major components were given in the presentation.

The major elements of the VF include:

<u>Primary Barriers - Passive Confinement</u>

Stainless Steel Liner on Walls and Pit
Walls, Roof, and Mat
Hatches
Special Doors
Shield Windows
Pipe Penetrations
Pit Embedded in Tight Clay and Glacial Till

<u>Secondary System - Active Confinement</u>

Primary Filter
Secondary Filter
Fan blower
HVAC Ducts
Stack
Standby Generator
HVAC Controls

<u>In-cell Equipment</u>

Process Vessels and Piping
Melter Supports

Coolers
Overhead Crane
Canister Transfer System
Canister Turntable
Canister Storage Rack
Canister Transfer Cart

Complete reports on some of the items are available and have been reviewed (Gates et al., 1989; Gates, 1989c; Gates, 1992; Gates and Semple, 1992). Some additional information was provided in this presentation on the safety classification system used for the WVDP, as well as hazard summaries (Figure 3-1). No new information on the civil structure and special doors seismic analysis or the shield window tornado projectile penetration was presented.

Open questions are given in Figure 4-7 (Gates, 1993). The results of the stack report were summarized (Question 1). This should be considered a closed issue. The important parameters concerning the canister cart (Question 2) are: (i) it is designed such that it will not topple under normal operation; (ii) the probability of the cart being in close proximity to the primary filter in the event of a DBEQ is very remote; and (iii) the use of redundant primary filters provides a backup. Based on presented results (Gates, 1993), this should be considered a closed issue.

The amount of steel to limit thermal cracking in the VF walls was considered a nonissue because they are in excess of those given in the appropriate ACI codes (Question 3). This also should be considered a closed issue. Results of the seismic separation between the VF and EDR were presented again with no new information (Question 4). Again the question of the appropriate damping in the soil elements arises. Results associated with the aged characteristics of the RodoFoam were stated to have been obtained, but no detailed information was presented.

The margin of safety formulation (Question 5) is outlined in sufficient detail (Figure 3-7). The question associated with soil damping in the elastic half-space is still an open issue. Indications were given that additional analysis is being performed to verify the assumptions made. It will be necessary to review the results of this analysis before this item can be closed. A brief verbal discussion of the thermal stress analysis in the VF wall (Question 8) was given. Indications are that the stress levels are very low. This information needs to be summarized in written form prior to closure of this issue.

The major new information presented in this report was associated with the component review and margins analysis. This included the:

— Melter Supports (Overview with no results),
— Canister Storage Rack and Waste Header Guard (Overview with mode shapes. Stress ratios less than 0.66) Figure 3-6
— Overhead Coolers (Overview with no results)
— Overhead Cranes (Verbal indication of possible uplift)
— HVAC System (Overview of system)

QUESTIONS FROM PAST MEETINGS AND REPORT REVIEWS BY NRC

1. Stack collapse potential on VF

2. Toppling of Canister Transfer Cart

3. Thermal steel percentage in VF Walls

4. Seismic Separation between VF and EDR

5. Definition of Safety Margins - Doors Report

6. Secondary effects of Tornado and Earthquake on VF barrier integrity

7. Soil Damping in Elastic Half Space Model of VF

8. Methodology followed in reviewing thermal stresses in VF wall -- hand analysis.

Figure 4-7. Questions from past meetings and report reviews by NRC (Gates, 1993)

— Primary Filters (Wire Frame Model with summary Margins of Safety. No details on calculations used to obtain Margins of Safety.)
— Secondary Filters (Seismic Qualification Testing. Successful with some structural modifications.)
— Secondary Filter Room Ducts and Supports (Overview with no results)

The basic procedures for analysis of these components are consistent with current practice. (The loads are based on response spectra utilizing the high values of soil damping.) The only indicated area of concern was with the uplift of the overhead crane during the design basis seismic event. Each of these issues needs to be reviewed in terms of the soil damping value and the resultant design spectra used for each piece of equipment.

- Donovan, Loyd E., February 24, 1993, "Equipment Seismic Qualification Program," West Valley Nuclear Services Co., Inc.

A brief overview of the qualification procedure for various equipment was provided by Mr. Loyd E. Donovan, WVNS. The presentation included overheads and two videos, generic impact testing, and qualification testing of the diesel generator. The equipment requiring qualification can be divided into two major categories:

Construction contractor-supplied equipment that is qualified by analysis and/or test included:

— HVAC Main Exhaust Blowers
— HVAC Damper and Butterfly Valves with their Actuators
— Diesel Generator Day Tank
— Motor Control Center and Electrical Switch Gear
— Uninterruptible Power Supply (UPS)
— HVAC Sensor Components and Solenoid Valves

Equipment furnished to the construction contractor that has been qualified by analysis and/or test included:

— Primary Filter Housing
— Secondary Filter Housing
— HVAC Control Panel
— Standby Diesel Generator
— In-cell Coolers
— In-cell Light Mounts
— In-cell Remote Camera Mounts

The presentation was an overview of the program and no attempt was made to provide any information of the results of the analysis and/or testing. The basic approach of the equipment qualification program is in accordance with established procedures. It will be necessary to review reports as they become available to check the analysis and/or testing results, if this is to be considered part of the program.

4.1 DOCUMENTS YET TO BE RECEIVED

In addition to the materials identified above, it is expected that further information will be required to complete the CNWRA review. Some of this information may be obtained through coordination meetings on an as-needed basis from appropriate participants in the WVDP project. In the August 7, 1992, letter from the DOE to the NRC, the following documents were identified as being in various stages of completion and not yet in final form for transmittal to the NRC.

High-Level Waste Transfer System (HLWTS) and VF Soil Structure Interaction Analysis

Piping and Trench Seismic Design

Interpolated Solids Data

Simplified Structural Interaction Analysis (EBASCO) (Soil-Structure)

Dames & Moore Review of Structural Interaction Analysis

Canister Transport System Seismic Analysis

Canister Transport System Design Calculations

In-cell Equipment Seismic Analysis

Construction Contractor's Analyses, Testing Techniques, & Calculations

WVNS Review of Construction Contractor's Analyses

Dames & Moore Review of Selected Analyses

EBASCO Services, Inc., June 12, 1987, "Vitrification Facility, Civil Design Criteria," WVNS Reference No. EBAR 837A.

5 REVIEW/ASSESSMENT

The CNWRA review of the structural evaluation of the VF system in terms of the DBEs (including Earthquake and Tornado) for the major elements is summarized in Table 5.1.

Complete reports on some of the items are available and have been reviewed, as discussed in Section 4. In the Table 5.1, NA refers to Not Applicable and NR refers to Not Reviewed. In terms of NA, the loading condition does not affect the corresponding specific components. For example, in-cell equipment will not experience the wind loading since it is protected by the primary structure of the cell. For those items listed as NR, no information was provided for use in the review process.

For those elements that were reviewed, the status is considered to be either open or closed. To be closed, sufficient information had to be presented in terms of the loading, modeling, and results to provide confidence that the element will withstand the DBE. Based on this review and currently available information, the design of these items is considered adequate.

For the open items, either the analysis or testing of this element is yet to be completed and results reported or a number of questions need to be resolved prior to closure. A number of the documents reviewed in Section 4 presented analyses and testing that were in progress, and the results of these will have to be reviewed. For others, analyses or testing were complete, but major questions concerning assumptions made during the analysis procedure remain.

The major remaining open issue is the value used for soil damping in the various models. Changes in this value will propagate throughout all analyses and testing performed for seismic qualification of the various components. The 20 percent soil damping value used in past analyses is not justified in any of the reports reviewed. Twenty percent damping implies nonlinear soil behavior and a degree of soil failure. The consequences of soil failure are not discussed in the reports. Once an acceptable value of damping in the soil is defined, the margin of safety of the various major elements can be checked. As a first approximation, it is not necessary to redo the entire analysis; the margins of safety can be adjusted by the ratio of the damping values. Similarly, for seismic qualification testing, the required response spectra can be adjusted and compared to those generated during testing, which are oftentimes conservative.

A second major open issue associated with the seismic DBE is the definition of the earthquake loading condition in terms of the required response spectra. For the majority of the results presented, the horizontal levels were defined in terms of a deterministic approach based on a standard RG 1.60 spectra with a peak acceleration of 0.1 g. This excitation was applied as independent signals in two perpendicular horizontal directions. From information reviewed, the vertical spectra was defined as two-thirds of the horizontal spectra as specified in RG 1.60. For east coast events, based on recorded events in Canada on soil, this two-thirds reduction may not be applicable. The SSE for nuclear power plants in the eastern U.S. are undergoing re-evaluation. This may imply an upward revision for the present 0.1 g DBE. Currently available results will have to be interpreted with respect to updated definitions of the DBE for the seismic event.

Table 5.1 Components Reviewed in VF

Primary Barriers — Passive Confinement		
Component	**Seismic**	**Tornado**
Stainless steel liner on walls and pit	Open (Damping)	NA
Walls, roof, and mat	Open (Damping)	Closed (Empirical)
Hatches	Open (Damping)	NR
Special doors	Open (Damping)	Closed (FEA & Empirical)
Shield windows	NR	Closed (Worst Case)
Pipe penetrations	NR	NR
Pit embedded in soil	Open (Damping)	NA
Secondary System — Active Confinement		
Primary filter	Open (Damping) — In Analysis	NR
Secondary filter	Open (Damping) — Tested	NR
Fan blower	NR	NR
HVAC ducts	NR	NR
Stack	NR	NR
Standby generator	Open (Damping) — Tested	NR
HVAC controls	NR	NR
In-cell Equipment		
Process vessels and piping	NR	NA
Melter supports	Open (Damping) — In Analysis	NA
Coolers	NR	NA
Overhead crane	Open (Damping)	NA
Canister Transfer System		
Canister turn table	Open (Damping) — In Analysis	NA
Canister storage rack	Open (Damping) — In Analysis	NA
Canister transfer cart	Open (Damping) — In Analysis	NA
Secondary Structures		
EDR Building	Open (Damping)	NA
Stack	Open (Damping)	Closed (Analysis)

NA: Not Applicable
NR: Not Reviewed

5.1 SEISMIC ANALYSIS USED IN REVIEWED DOCUMENTS

5.1.1 Definition of Loading

For the analysis and testing associated with the civil structures and equipment at the WVDP, the seismic DBE is defined in terms of the required response spectra. From a review of documentation, the deterministic horizontal DBEQ of the primary confinement structure is defined as a 0.1 g peak ground acceleration in the free field, utilizing the NRC RG 1.60 spectra. The RG 1.60 spectra is based on an ensemble average of a large number of seismic events recorded on soil and rock sites and represents the mean plus one standard deviation level, normalized to 1.0 g peak acceleration (Figure 5-1). Adaptation to a specific site requires adjusting the amplitude of the entire spectra to the desired peak acceleration level.

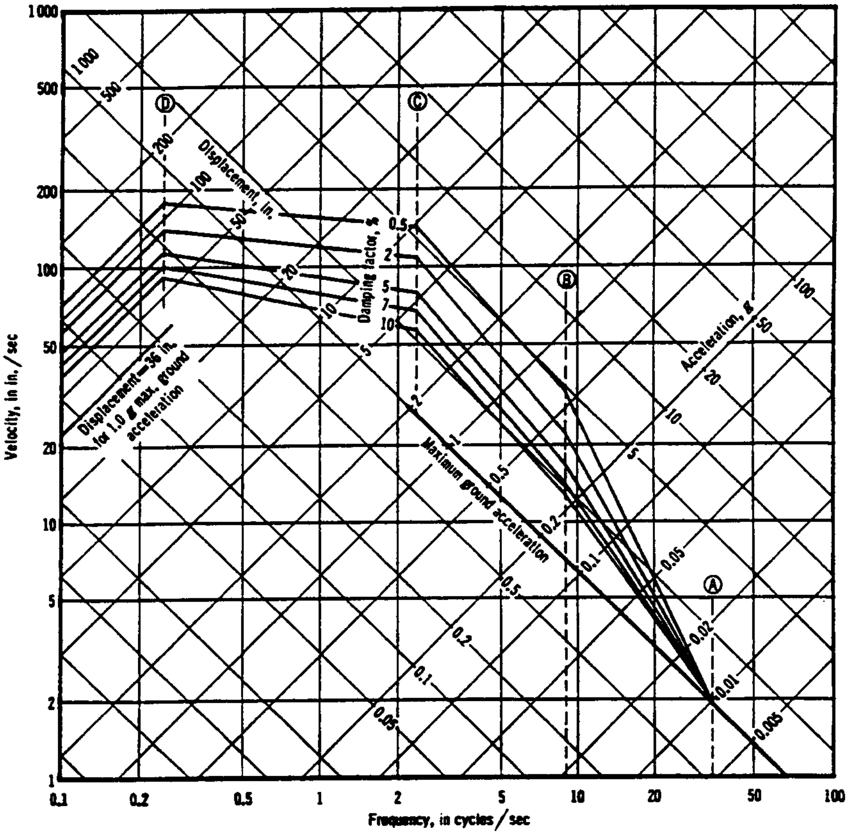
For the Dames & Moore deterministic analysis of the primary confinement structure (Gates, 1993), the vertical spectra is specified as two thirds of the horizontal spectra. Gates presents data from an EBASCO (1987) report which is not available. For facilities east of the Rockies, based on the results presented in Paragraph 3.2.9.4 of ASCE (1980), the vertical acceleration spectra between 3.5 and 33 Hz should be equal in intensity to the horizontal component. This is consistent with the RG 1.60 required spectra (Figures 5-1 and 5-2). Based on a review of the EBASCO design spectra calculated from the time histories, they are consistent with the definitions given in RG 1.60.

Current probabilistic practice requires a process of tailoring, whereby the definition of the seismic event is based on either measured data or expert opinions adjusted to the specific site in question. This includes a definition of both the frequency content and peak acceleration of the seismic event. The frequency content is often based on analysis of recorded seismic waves, including a model of the geologic fault and attenuation with distance from the fault. The safe shutdown earthquake (SSE) levels, although deterministic, may be typically defined in terms of the probability of a given level that will occur 10^{-3} to 10^{-4} times per year. Information on the seismic criteria for a number of nuclear power plants in northwestern New York was presented by Dames & Moore. For the four units presented, the design or safe shut-down deterministic g values ranged from 0.12 to 0.15 g, greater than the 0.1 g utilized for WVDP.

Recent information, from studies by the Electric Power Research Institute (EPRI, 1988) and Bernreuter et al. (1989), indicates that the mean values for peak acceleration for events with a 10,000 year return period for nuclear power plants in this area is 0.14 g for rock sites and 0.21 g for shallow soils. Note that these g values are mean 10^{-4}/yr peak values. There is some concern over the standard deviation values determined in these two studies, based on a comparison of the median value with the mean plus one standard deviation.

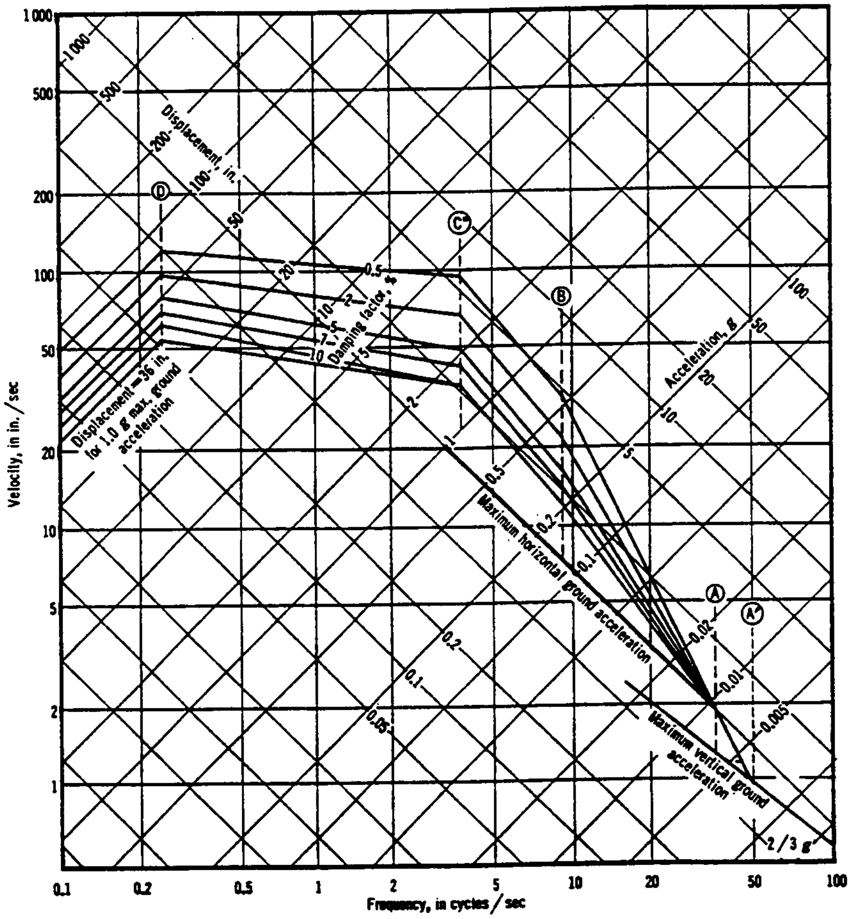
These deterministic and probabilistic SSEs would indicate that the WVDP analysis may be nonconservative, although the short duration of operation of the VF and its different function need to be taken into account.

In general, the use of the RG 1.60 spectra is consistent with established procedures. The question that remains is, what is the peak acceleration to be used, based on the site and short operating period of the facility?



Percent of Critical Damping	Amplification Factors for Control Points			
	Acceleration			Displacement
	A (33 cps)	B (9 cps)	C (2.5 cps)	D (0.25 cps)
0.5	1.0	4.96	5.95	3.20
2.0	1.0	3.54	4.25	2.50
5.0	1.0	2.61	3.13	2.05
7.0	1.0	2.27	2.72	1.88
10.0	1.0	1.90	2.28	1.70

Figure 5-1. Horizontal design response spectra from RG 1.60 and ASCE No. 58 (RG 1.60)



Percent of Critical Damping	Amplification Factors for Control Points				
	Acceleration				Displacement
	A (50 cps)	A′ (33 cps)	B (9 cps)	C′ (3.5 cps)	D′ (0.25 cps)
0.5	0.67	1.0	4.96	5.67	2.13
2.0	0.67	1.0	3.54	4.05	1.67
5.0	0.67	1.0	2.61	2.98	1.37
7.0	0.67	1.0	2.27	2.59	1.25
10.0	0.67	1.0	1.90	2.17	1.13

Figure 5-2. Vertical design response spectra from RG 1.60 and ASCE No. 58 (RG 1.60)

5.1.2 Structural Modeling and Analysis Procedures

5.1.2.1 Simplified Strength of Materials

A conservative approach to the seismic analysis is to perform a simplified equivalent static analysis based on the total weight of the structure and the peak acceleration of the DBE shock spectra. This procedure can be utilized for structures and equipment that have a first mode frequency greater than the cutoff frequency for the elevated portion of the response spectra. This corresponds to 33 Hz for the horizontal RG 1.60 spectra and 50 Hz for the vertical RG 1.60 spectra with the two-thirds factor applied.

The static procedure can also be applied to structures whose first mode is less than the cutoff frequency for the elevated portion of the response spectra. In this case, the equivalent static load for a given mode is equal to the weight of the structure times a factor including the peak acceleration and an amplification factor, taking into account damping of the mode in question. For a single mode, this equivalent static load is added directly to all other loads to determine the margin of safety. If multiple modes exist, several different modal combination procedures can be applied. Use of the equivalent static procedure for structures with more than one mode is difficult, and an FEA procedure is recommended.

It appears that this equivalent static procedure was utilized for a number of elements, including the doors, under the seismic loading. Verification that the first mode frequency for the doors is greater than the cutoff frequency of the response spectra has not been given in the reports reviewed. This must be done to verify the loadings utilized.

5.1.2.2 FEA Procedures

For complex structures and equipment with modes within the elevated region of the response spectra, it is advisable to perform an FEA. The structure or equipment is first modeled using finite elements to define the appropriate geometry mass, stiffness, and damping characteristics. These models can range in complexity from detailed 3D solid models to simpler 3D beam and truss models. For static analysis, after defining the structure, loading, and restraints, the response of the structure can be calculated. For dynamic analysis, an eigenvalue analysis is first performed to determine the natural frequencies and mode shapes. These results can then be used to determine the dynamic response under a variety of loading conditions.

It should be noted that FEA procedures are appropriate where the functionality of the structure or equipment can be defined in terms of the response motions and resulting stresses in the structure. If operability is required, it is often easier to perform testing to verify functionality under the dynamic loading.

Dynamic analysis procedures for determination of the structural response of critical elements to earthquake events can be based on a time-history solution or a response-spectra solution using modal superposition. The analysis for this program utilized a time-history solution based on a single, 10 second long time history with three statistically independent signals for the two horizontal and one vertical directions. This excitation was applied to a variety of FEA models of various critical structures and equipment.

There are several general concerns associated with the seismic analysis procedures utilized. The first is the use of only a single time history to calculate the dynamic response of the system. It is common practice to utilize multiple time histories to develop an envelope for the response of the system. The requirement for use of multiple time histories is based on the fact that the development of a time history from a response spectra is not unique. There are an infinite number of time histories that can be shown to envelope a given response spectra. Each time history will produce a different response of the system. To limit these, several parameters are often specified: duration of the strong motion portion of the event, the shape of the Power Spectral Density (PSD) of the signal, as well as the peak to root mean square (rms) ratio during the strong motion.

To ensure that the actual seismic event is modeled properly, multiple time-history solutions are often performed and the envelope of the responses and stresses due to these time histories calculated. This envelope will then provide some confidence that resulting margins of safety are conservative. It is also possible to multiply the required spectra by some margin, nominally 10 percent, to account for the variation. It appears that this was done for this program.

The other approach is to utilize a shock spectrum analysis which incorporates the statistical nature of the problem directly. For this case, the load is defined in terms of the shock spectrum only and a modal superposition procedure utilized to determine the system response. Both the enveloping of multiple time histories and the shock spectrum approach have been shown to produce similar results.

A second concern is the duration of the signal utilized in the analysis. Typical representation of an earthquake is a total of 30 sec duration with 15 sec of strong motion. The duration is dependent on the level of the seismic event as well as the geological structure in the local region. The necessity of utilizing a longer signal is dependent on the fundamental frequencies and damping of the system and the time it takes the response of the system to develop. For the calculated natural frequencies of 5 to 7 Hz and the assumed composite damping of 15 percent, the 10-sec duration will not be a problem. If the true damping is less or the system frequencies lower, the duration of the event may become more important.

This leads to the final question concerning the use of 15 percent or greater for the composite damping of the system. Twenty percent soil damping suggests nonlinear soil behavior, almost always hysteretic. Therefore, duration or number of cycles of loading is important. An arbitrary 15 sec of strong motion may not predict soil response accurately. As indicated previously, this high value is due to the contribution of radiational damping in the soil. To accurately define the strain field in the soil surrounding the foundation, it is necessary to develop a full 3D solids model of the system, including the soil and foundation structure. From discussions, it appears that this type of analysis is currently in progress, but results have not been presented.

Results of this type of soil-structure model can be used for subsequent simplified analysis. The damping values used in the original 3D stick model were 7 percent for the structural elements and 20 percent for the soil, to account for both material and radiational damping. The review process used by Dames & Moore utilized a composite modal damping of 15 percent, based on the relative strain energy in the elements. The value of damping used for the structural elements is consistent with the damping values given in RG 1.61 for reinforced concrete at the DBE level.

The specified seismic DBE of 0.1 g peak acceleration is likely to produce overall strain levels in the soil and structure that are low. As a more conservative approach, it is recommended at this time that no more than 15 percent damping be utilized for the soil elements. In addition, a more conservative approach would be to utilize a structural damping value of 4 percent based on the OBE level for reinforced concrete structures. The strains in the concrete as a result of the 0.1 g earthquake are likely to be more closely related to the OBE responses given in RG 1.61.

Procedures used to develop results during the FEA are straightforward. The only area of question is the method used for combination of the modes for a given direction or the combination of results for each of the directions. Model and direction combination procedures are not given in the reports reviewed. Typically, these are done using a square root of the sum of the squares (SRSS) approach. Combining the results from various loads, as required, and comparing these to allowables is used to develop the margins of safety. Multiple modes and input directions are applicable to the WVDP.

5.1.2.3 Test Procedures

For equipment whose functionality is not directly related to peak motion or stress, it is often easier to qualify it by testing. Typically, a test program includes subjecting the equipment to time histories that envelope the required response spectra with a margin applied, typically 10 percent over the entire frequency range. The excitation is applied in three directions, either three, two, or one axis at a time. The functionality of the equipment is checked before, during, and after the event, as required. If the equipment functions properly under all conditions, it is assumed to be qualified.

The basic approach of the equipment qualification program is in accordance with established procedures for nuclear power plant equipment. It will be necessary to review reports as they become available to check the testing results. An important consideration is the definition of the required response spectra for the testing. If it is based on the utilization of the high value of soil damping, it may be necessary to revisit this issue and develop new elevated response spectra based on more conservative estimates of the damping. It may happen that the actual test spectra will also envelope these update elevated response spectra.

5.1.2.4 Results and Margins Assessment

Primary Civil Structures

For the VF primary structure using the 3D FEA stick model (Figure 3-3), results were presented in Gates (1993). Results are in terms of modal frequencies, peak displacements at various locations, elevated response spectra, and forces and strains in the elements. The response spectra indicate a primary response at 5 to 7 Hz, corresponding to the first natural frequency of the VF cell, which is consistent with reinforced concrete structures of this geometry.

Analysis of the primary confinement structure response included the roof, walls, mat, soil, special doors, hatch, and shield windows. Margins of safety for each of these elements are given in Figure 5-3. The failure mode for the cell roof and walls is yield of the reinforcing steel, resulting in a plastic hinge. For the foundation mat, the failure mode was uplift, while the soils failure was based on cracking. Each of these values are acceptable based on the assumed loading and damping in the soils. If adjustments to the soil damping or level of excitation are made, it may become necessary to reevaluate the values.

Indications are that cracking of the roof slab and the top portion of the walls of the VF could occur during an earthquake of magnitude less than the DBE, when out-of-plane bending is combined with

MARGINS OF SAFETY
VF PRIMARY CONFINEMENT BARRIERS

	DBEQ Earthquake	DBT Missile	Thermal Yield
Roof	>3.4	> 6	4.3
Walls	>3.5	>6	2.5
Mat	>6.5	>10	—
Soil	>6	—	—
Special Doors	>1.5	>1.9	—
Hatch	>6[2]	>6[2]	—
Shield Windows	>2	(1)	—

(1) If window is penetrated by missile, consequential exposure is below acceptable limits.

(2) With modification.

Figure 5-3. Margins of Safety for VF primary confinement barriers (Gates, 1993)

thermal stresses. The assumption made in this case was that cracking is not considered a failure because there will be no potential for release of radiological materials since cracking is not likely to propagate through the thickness of the structure. It should be noted that an analysis based on the cracked structure should be performed to define the worst-case condition.

Doors

The structural design review and integrity assessment of the eight special doors of the VF that function as critical elements of the primary confinement barrier system were part of the Dames & Moore review. Operability and/or leak prevention is not a requirement for the special doors, only that they remain in place.

Three separate methods of investigation were used to assess the structural integrity of the shield doors under extreme environmental conditions. These included analytical methods, experimental methods, and engineering judgement. A comprehensive list of failure modes for the doors was developed (Gates, 1992).

The combined earthquake and dead load demand on the doors and their support hardware was based on using the peak response for the spectra provided in WVNS-EQ-264. The critical margin of safety for the doors was given as 1.5 (Figure 5-3). Increasing the loading due to changes in soil damping and seismic peak acceleration could result in unacceptable margins of safety based on the conservative analysis performed. It may be necessary to perform more complete dynamic analysis of the door systems to get a true indication of their strength characteristics.

Equipment

Very few detailed results were given on the specific pieces of equipment. It appears that the majority of this work is currently in progress. Presentation of the component review and margin analysis for in-cell and ex-cell equipment were included in Gates (1993).

The basic procedures used for analysis of these components are consistent with current practice. Each of these issues needs to be reviewed in terms of the soil damping value and the resultant design spectra used for each piece of equipment.

Other Civil Structures

The behavior of the existing 160-ft Reprocessing Plant Ventilation Stack (Figures 3-4 and 4-2) was analyzed under extreme environmental loads that include wind, tornado, and earthquake. The intent of the study was to realistically assess the collapse potential of the stack and the risk that such collapse might pose to the VF. Even if this failure were to occur, the possibility of any damage to the VF is remote. Although the resulting margins of safety are given, no calculations are shown supporting these. It would be appropriate to obtain some measured data on the dynamic characteristics of the stack, as well as major elements in the VF, for verification of the analytical models.

In addition to structural failure, the potential for contact between the VF cell and the EDR of the existing Nuclear Fuel Reprocessing Plant (Figure 4-2) was calculated. The separation between the VF and the EDR during the seismic events is based on modeling the response of the building under the influence of traveling seismic waves combined with the dynamic response of the buildings. This analysis showed a maximum of 0.8 in. closure for a total gap of 3 in., giving an acceptable margin of safety. Reduction of the soil damping to 10 percent could increase the deflection by a factor of 2, with a corresponding reduction in the margin of safety.

5.2 TORNADO ANALYSIS USED IN REVIEWED DOCUMENTS

5.2.1 Definition of Loading

The maximum design tornado utilized the following parameters:

Maximum Wind Speed	160 mph
Tornado Radius	150 ft
Rotational Wind Velocity	110 mph
Translation Wind Velocity	50 mph
Pressure Differential	0.35 psi
Pressure Transient	0.15 psi/sec
Projectiles	Wooden plank at 85 mph
	Steel pipe at 50 mph

These values are consistent with current practice (ASCE No. 58).

5.2.2 Structural Modeling and Analysis Procedures

The structural response to wind loading as well as the penetration potential due to tornado-borne missiles were considered. Wind loading and pressure differential are of secondary concern for the major structural elements, but are critical for the design of the HVAC system. Analysis of the response of the system to tornado and the maximum wind loading is based on the procedures of ANSI A58, "Minimum Design Loads for Buildings and Other Structures." The penetration resistance of the primary structural elements was based principally on comparison to published data for concrete, steel, and soil barriers. Some analysis of the doors was performed to consider rebound.

The methodology for tornado missile load analysis on the special doors is based on a two-step approach: (i) equivalent static or (ii) dynamic response. This is common engineering practice. Based on discussions in the text of this report (Gates, 1992), all analysis was done in terms of the equivalent static analysis, although no supporting calculations are shown.

For the shield windows, the approach taken was to assume that penetration did occur and determine the level of leakage. If the leakage rate was acceptable then the design was considered to be acceptable. This procedure was discussed in detail in Section 4.

5.2.3 Results and Margins Assessment

Results of the margins assessment for the primary civil structures are given in Figure 5-3. The major structural elements, including the roof, wall, and mat, have margins of safety greater than 6. This is not the controlling factor for design of these structures. The special doors on the exterior of the facility need to only stop tornado missiles and remain in place. After redesign of certain critical elements, all the doors have acceptable margins of safety for the tornado loading. For the shield windows, the amount of leakage through the most credible hole was found to be acceptable by Dames & Moore (Gates et al., 1989).

5.3 OTHER LOADINGS USED IN REVIEWED DOCUMENTS

Only limited discussion and documentation was presented on other loading conditions, such as dead load, live loads, and thermal loads. Response of the structure to the thermal loads for the roof and wall was based on simplified calculations and showed acceptable margins of safety (Figure 5-3). In most cases, by following the ACI procedures for determination of the amount of steel reinforcement in concrete, this loading is not critical. The other loading conditions are taken into account when using the appropriate combinations given in the next section.

5.4 COMBINED LOADING USED IN REVIEWED DOCUMENTS

For determination of the structural integrity of the various components, the following design loads were considered individually or in combination:

Dead Load (D)
Live Load (L)
Thermal Load (T_0)
Soil Pressure Load (H_{Static}, $H_{Dynamic}$)
Wind Load (W)
Tornado Load (T_t)
Seismic Load (E_{SSE})
Differential Settlement ([Δ])
Internal Pressure (P_0)
Load from Earthquake, Uniform Building Code (E_{ubc})

For this program, Dames & Moore indicated that the combination of procedures outlined in Figure 3-2 were utilized to determine the margin of safety (Figure 3-7). For the results presented, it was not possible to determine if these procedures were followed, since only the final margins are given. Margins of safety for the individual components are not given.

6 RECOMMENDATIONS

As part of the February 1993 presentation by W.E. Gates of Dames & Moore, a number of questions from past meetings and report reviews by the NRC were given (Figure 4-7). In addition, Table 5-1 summarizes the structures and equipment that have been considered under the DBEs for seismic, tornado, and wind loading. As can be seen from that table, results on a number of elements are not available for review at this time. Prior to recommending acceptance of the design package, additional review will be required.

Resolution of the following two major issues needs to be made at the earliest possible time, since they propagate throughout the remainder of the open issues.

(i) Definition of the DBE seismic event is in terms of the peak acceleration value and the associated response spectra tailored to the WVDP site needs to be justified. Currently, all analysis is based on a deterministic approach using a RG 1.60 spectra normalized to a 0.1 g peak acceleration. From the presentation (Gates, 1993), it appears that a 10 percent margin was applied to the peak acceleration for all analysis, to account for statistical variations between time histories. This is consistent with current practice for equipment. The issue is whether the peak acceleration level is appropriate for the WVDP site. As indicated earlier and based on a probabilistic approach and expert opinions (Bernreuter et al., 1989), sites in the region may have seismic events with a peak acceleration of 0.14 (rock) to 0.21 (deep soil) g. Note that these are mean values while the RG 1.60 specification is associated with the mean plus one standard deviation.

It is important to define the peak acceleration levels and response spectra for both the horizontal and vertical spectra that are consistent with the expected events in the region. This tailoring process is consistent with current NRC practice and should be carried out for this plant.

A large amount of analysis has been performed based on the 0.1 g levels and the RG 1.60 horizontal and vertical spectra (Figures 5-1 and 5-2). It is not necessary to re-analyze based on any updated level. For linear analysis, it is possible to apply a correction factor and develop new margins of safety for the seismic event. An additional factor that needs to be taken into account is the expected life of the plant, 5 years. This is less than the typical 30 to 40 years of a power plant and will influence the final determination of the overall margins of safety.

(ii) Justification of the damping value used for modeling the soil and structures throughout the program needs to be formalized. It is understood that a detailed model of the soil-structure interaction problem is currently being developed. Incorporation of the detailed nonlinear soil characteristics available from previous work into the model will result in a justifiable model of the soil. Care should also be taken in defining the level of excitation that is to be applied (see (i) above). Because of the highly strain-dependent nature of the soil, it will be necessary to ensure that the inputs are appropriate. Results of this analysis have not been reviewed to date.

In addition to the damping value for the soil, it is appropriate to review and provide justification for the amount of damping associated with the structure itself. The original analysis utilized a damping value of 7 percent for the reinforced concrete structure. Based on the level of excitation, the strain levels in the concrete members may not be sufficient to realize this level of damping. It may be more appropriate to use the 4 percent value defined in RG 1.61 associated with the OBE level for reinforced concrete.

As indicated earlier, the definition of the damping levels will influence the results in a number of areas. In addition to the analysis of the structures themselves, it needs to be reflected in the elevated response levels specified for qualification of equipment. This includes all analysis as well as testing. Again, it may be possible to apply a correction factor to the existing analysis to develop the updated margins of safety.

These outstanding issues will be reviewed as part of the VF SAR.

The review of information available at the CNWRA addressed the majority of questions given in Figure 4-7 (Gates, 1993). These included:

The results of the stack report were summarized (Question 1). For this structure, the probability of collapse due to the seismic, tornado, and wind DBE is small. Even if collapse occurs, there would be no significant damage to the crane maintenance room (CMR). The stack would not come into contact with the VF Cell roof or wall. This should be considered a closed issue as associated with the VF Cell.

The important parameters concerning the canister cart (Question 2) are: (i) it is designed such that it will not topple under normal operation; (ii) the probability of the cart being in close proximity to the primary filter in the event of a DBE is very remote; and (iii) the use of redundant primary filters provides a backup. Based on these conclusions, this should be considered a closed issue.

The thermal steel percentage in the VF walls was considered a non-issue because it is in excess of that given in the appropriate ACI codes (Question 3). This should be considered a closed issue.

Results of the seismic separation between the VF and EDR were presented (Question 4). Results associated with the aged characteristics of the RodoFoam were obtained, and indications are that variations in properties do not significantly affect the results, although no detailed information supporting this was presented. Again the question of the appropriate damping in the soil elements arises. This issue needs to be reviewed again once the two questions given above are answered with justification.

The margin of safety formulation (Question 5) was outlined in sufficient detail (Figure 3-7). The question associated with soil damping in the elastic half-space is still an open issue. It will be necessary to review the results of this analysis before this item can be closed.

Question 6 refers to failure of the reprocessing plant stack and its influence on the VF. Based on the results presented (Gates et al., 1989) this should be considered a closed issue. Question 7 is addressed in detail in (ii).

A procedure for thermal stress analysis in the VF wall (Question 8) has been presented verbally. Indications are that the stress levels due to differential thermal loading are very low. This information needs to be summarized, in conjunction with Question 3, in written form, prior to closure of this issue.

In general, the basic procedures used for seismic analysis of the civil structure and equipment are consistent with current practice. Each of these issues needs to be reviewed in terms of the soil damping value and the resultant design spectra used for each structure and piece of equipment. In the final presentation of the margin of safety, it is appropriate to present the component stress levels, i.e., dead load, live load, pressure load, etc, in addition to the composite values. This will be extremely useful in determining the effects of changes in such parameters as damping and input levels.

The basic approach of the equipment qualification program is in accordance with established procedures. It will be necessary to review reports as they become available to check the testing results. Resolution of the damping and input seismic level questions will have to be made prior to the final review.

7 EXECUTIVE SUMMARY OF CONCLUSIONS

7.1 SEISMIC DBE

There are two major issues associated with the review of the seismic analysis and testing that must be resolved at the earliest possible time. The first is a definition of the peak acceleration levels to be used for both the horizontal and vertical directions based on tailoring to the WVDP site. The second is the damping values to be utilized for both the civil structure and the soil elements in the 3D FEA models.

Resolution of these items will allow judgements to be made on the seismic adequacy of the primary containment civil structures, including the doors and shield windows, as well as equipment.

The following items need to be reviewed based on the answers to the damping and acceleration level questions:

(i) Breakdown of the Margins of Safety calculated for each element in terms of their basic parts: dead load, live load, operating pressure, operating temperature, hydrostatic pressure, static soil pressure, DBE, etc. The relative magnitude of each of the parts will allow for adequate judgement of the design to function properly before, during, and following the seismic event.

(ii) Results of the equipment qualification analysis and testing performed. The results should include the critical elements defined in Table 5-1.

These outstanding issues will be reviewed as part of the VF SAR.

7.2 TORNADO AND WIND DBE

The resistance of the primary containment civil structures and the special doors and shield windows to penetration by the tornado-borne missiles should be considered a closed issue. Based on the best available information, these elements are sufficient to ensure that complete penetration and subsequent release of radiological hazards will not occur.

Design review reports by DOE contracts on the capabilities of the applicable structures and equipment to withstand the tornado and wind DBE pressure loading need to be reviewed. These are given in Section 8.2 of this report. This review would close out all issues associated with these two DBE.

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9 NOMENCLATURE — LIST OF ACRONYMS AND ABBREVIATIONS

3D	three-dimensional
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
CMR	crane maintenance room
CNWRA	Center for Nuclear Waste Regulatory Analyses
CTS	Canister Transfer System
DBE	Design Basis Event
DBEQ	Design Basis Earthquake
DBT	Design Basis Tornado
DC	Design Criteria
DOE	U.S. Department of Energy
EDR	Equipment Decontamination Room
EPRI	Electric Power Research Institute
FEA	finite element analysis
g	gravitational acceleration force
HLW	high level waste
HLWTS	High-Level Waste Transfer System
HVAC	heating, ventilation, and air conditioning
LLW	low-level waste
NMSS	Nuclear Material Safety and Safeguards
NRC	U.S. Nuclear Regulatory Commission
OBE	operating bases earthquake
PSD	Power Spectral Density
PUREX	Plutonium-Uranium Recovery EXtraction
RC	resonant column
RG	Regulatory Guide
rms	root mean square
SAR	Safety Analysis Report
SER	Safety Evaluation Report
SRSS	square root of the sum of the squares
SSE	safe shutdown earthquake
STS	Supernatant Treatment System
SwRI	Southwest Research Institute
THOREX	THOrium Recovery EXtraction
TRU	Transuranic
UPS	Uninterruptible Power Supply
VF	Vitrification Facility
VP	vitrification process
WSS	Waste Solidification Systems
WVDP	West Valley Demonstration Project
WVNS	West Valley Nuclear Services Co., Inc.